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2. A 50-word summary must accompany the paper, a 300-word synopsis must precede it, and a conclusion must end it.

3. The manuscript (an original and two duplicate copies) should be double-spaced on one side of 8½-inch by 11-inch paper. Three copies of all illustrations, tables, etc., must be included.

4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

5. Mathematics are recomposed from the copy that is submitted. Because of this, it is necessary that letters be drawn carefully, and that special symbols be properly identified.

6. Tables should be typed (an original and two duplicate copies) on one side of 8½-inch by 11-inch paper. Specific illustrations and explanation must be made in the text for each table.

7. Illustrations must be drawn in black ink on one side of 8½-inch by 11-inch paper. Because illustrations will be reproduced with a width of between 3-inches and 4½-inches, the lettering must be large enough to be legible at this width. Photographs should be submitted as glossy prints. Explanations and descriptions must be made within the text for each illustration.

8. Papers should average about 12,000 words in length and must be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

9. Technical papers intended for publication must be written in the third person.

10. If letter symbols are used in the paper they should be defined where they first appear, in the illustrations or in the text, and arranged alphabetically in an Appendix.

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AT,CO,CP,PL,PP,SU.

Journal of the
CONSTRUCTION DIVISION
Proceedings of the American Society of Civil Engineers

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(over)

DISCUSSION

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SOIL FREEZING TO RECONSTRUCT A RAILWAY TUNNEL

By George J. Low[1], A.M. ASCE

SYNOPSIS

Allowing for track rearrangements in a railway tunnel in Montreal (Canada) called for considerable ingenuity on the part of the contractors. The job was made especially difficult by the maze of underground facilities, and the fact that the work had to be performed under the street between two buildings. The subsurface conditions were such that freezing of the soil was resorted to. The problems attendant to such a construction procedure are outlined.

THE PROBLEM

The northern access to the Canadian National Railways' station in downtown Montreal is a double track tunnel 3 miles long under Mount Royal. It was built during the World War I, using a double concrete arch system to support the overburden near the portals. Farther inside, the tunnel consists of a single arch cut into the rock of the mountain itself. Hitherto, all switching from one track to another of the twin-track system had been done either within the station yeard itself or at a double cross-over switch well within the tunnel.

Track re-arrangements to install further switches just inside the tunnel portal were precipitated by the construction of a 42-story building straddling the railway facilities. This required the removal of the double arch and its dividing wall for the first 175 ft back from the portal and its replacement by a single arch spanning both tracks.

The physical difficulties involved were considerable. Not only did the affected portion of the tunnel run underneath a busy city street with its attend-

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ant maze of sewers, conduits, and pipes, but it was located between two buildings. One was a modern 9-story reinforced concrete building founded on till, whereas the other was an old, cracked, 3-story wood and brick building whose basement level was actually lower than its foundations that were in a silt layer. Since neither building was founded on rock, any excavation between them could cause settlement and damage.

Borings revealed that the limestone bedrock 30 ft below street level was overlain by a layer of dense dark gray till (about 8 ft to 10 ft thick), common in this area, and by a 6 ft to 8 ft thick stratum of silt. Above the silt was a layer of stiff dark brown clay forming the sub-base of the street. The silt was of a fairly uniform particle size whose grading curve showed 87% in the silt range. Compact in an undisturbed state, it exhibited marked dilatant properties. The water table lay above the silt that unfortunately, was the particular layer through which the new tunnel arch had to be advanced. Thus, if excavated while in this saturated state, the resulting seepage would have made the silt flow like a heavy liquid.

The problem to be solved before starting construction was how to excavate sufficient material to rebuild the tunnel roof safely and economically without disturbing the frequent train service or the stability of the adjacent buildings.

PLANNING

Since the first half of the tunnel length under consideration lay beneath a street intersection, there were no buildings to consider, and it could be done in the open using steel sheet piling and bracing. The few services in this area were diverted except for a 5-ft diameter concrete trunk sewer. Because it was in a poor condition, it was not diverted, but simply carried on a pair of steel trusses that were prestressed to avoid excessive deflection of the sewer. The struts against the steel sheet piling were also prestressed to resist the full existing pressure from the soil.

Examining the three main problems of the buried services, the building foundations and the plastic silt layer, several methods of construction for the critical length of the tunnel were considered.

For this critical section, the regular open cut method using steel sheet piling and bracing had two big disadvantages. It would not only cut through too many services, but would also require very heavy bracing to resist both the active soil pressure and the surcharge pressures from the buildings (Fig. 1). It was feared that the slightest deflection in the bracing would allow the two buildings to settle with unpredictable, but surely expensive, results. Underpinning the buildings was considered too difficult and too expensive.

Since the open cut method could not be used, the silt layer had to be stabilized and the tunnel advanced from the portal, working between the old arches below and the street above.

Because the permeability of the silt was low, its properties could not be changed by injecting either grouts, such as clay or cement, or chemical solutions such as sodium silicate or calcium chloride.

Unbalanced under pressure in silt, that causes the tendency to flow, can be balanced by compressed air. However, the busy railway tunnel precluded its use in this case.

The possibility of draining the silt by gravity, vacuum, or electro-osmosis was considered. Drainage by gravity through the tunnel roof was discarded because leaking water might not only cause trouble to the railway electric traction system, but also, the resultant ice formation (construction took place in winter) could cause serious obstructions to the regular passage of trains.

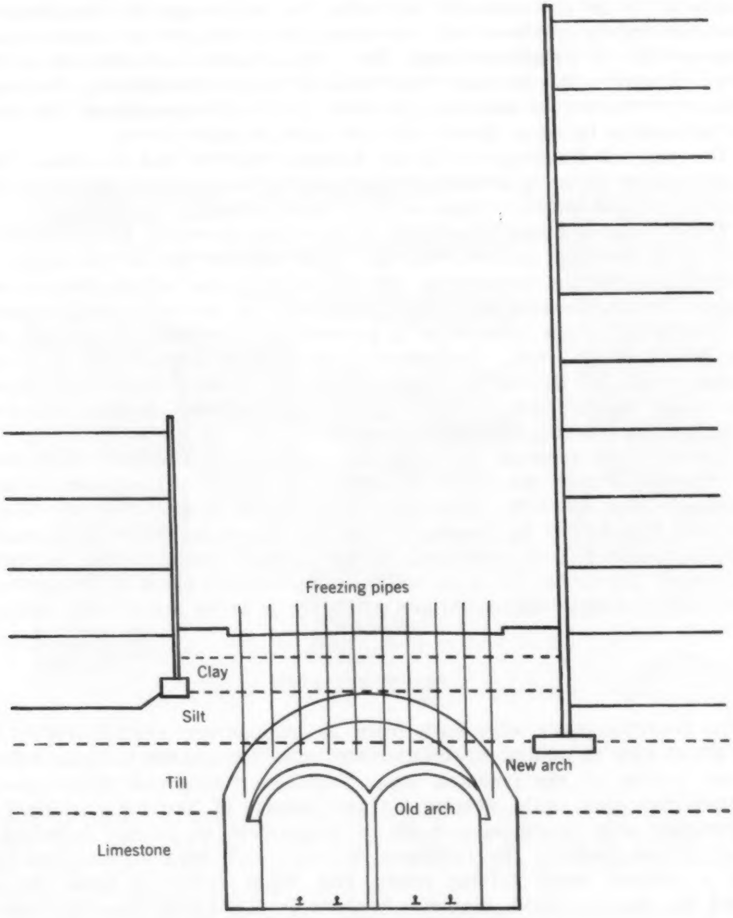

FIG. 1.—CROSS SECTION SHOWING POSITIONS OF SOIL LAYERS AND BUILDINGS ON EITHER SIDE OF THE TUNNEL.

Drainage by the use of a vacuum, such as in the wellpoint system, was thought inadvisable, since it would have been difficult to get the wellpoints into position, and the ring-main and pumps would have been about 21 ft above the bottom of the silt layer. To lessen the suction head, it would have

been necessary to criss-cross the site with trenches, seriously interfering with the buried services mentioned previously.

Drainage by electro-osmosis, involving the placing of electrodes, was next considered. If electrodes are placed in the gruund between drainage points, and a direct current is caused to flow between the electrodes and these points, making them anodes and cathodes respectively, an electro-osmotic force causes water to flow to the cathodes. The electro-osmotic permeability is the same for all soils, but in the case of silts, the gravity permeability is so low that, in comparison with it, the electro-osmotic permeability is relatively high. For this reason, this process is used in silts. However, the process was discarded after considering the possible damage to the buried services by electrolysis and possible interference to the installation by stray direct currents known to exist there.

The general disadvantage of any drainage method was the suspicion that leaking water pipes in the vicinity had supplied the water necessary to saturate the silt and would contniue to do so during attempts at drainage.

There remained the possibility of freezing the water in the silt by circulating a freezing solution through pipes driven into the silt layer. This method had decided advantages. The frozen material would not only act as support for the smaller building to maintain its pre-construction position, but also would allow excavation to proceed using regular tunnelling methods for a weak rock. Futhermore, the nominal spacing of the freezing points could be located to avoid buried services. Considering these big advantages against the disadvantages of inexperience in this type of construction, the freezing method was adopted.

Two different freezing methods were considered. The first was to freeze an arch of silt over the tunnel by installing a series of longitudinal pipes at suitable center-to-center distances. The second was to freeze a block of silt over the tunnel by means of vertical pipes driven from the roadway surface overhead. Both methods seemed feasible, but since the longitudinal arch pipes had to be bored in, whereas the vertical pipes of the block system could be simply hammered into position, the latter system was adopted.

CONSTRUCTION

The freezing pipes were made from $2\frac{1}{2}$-in. diameter seamless steel tubing, fitted with an interior 1-in. diameter pipe, leading the freezing solution to the bottom of the freezing pipe. Seamless tubing was chosen to avoid splitting the pipe while driving, as any leakage of freezing solution to the surrounding soil would have made it impossible to freeze, defeating the object of the method. To facilitate driving, each freezing pipe was fitted with a conical steel driving point. The whole system of pipes was connected by hoses to 3-in. diameter headers leading to and from the freezing plant.

To overcome air locks, inevitable in a parallel flow system, several air reservoirs and cocks were fitted at high points in the lines. Also, a crossover system was provided between the freezing plant and the headers, so that, by reversing the flow through the pipes, a stubborn air lock could be blown back to a suitable reservoir and outlet (Fig. 2).

The thermostatically controlled freezing plant consisted of two electrically driven compressors, circulating freon 12 gas through a heat ex-

changer. A methanol solution was, in turn, circulated through this heat exchanger before passing through the freezing pipes.

A methanol solution was chosen as the heat-carrying medium through the heat exchanger, since a strong solution of calcium chloride might have damaged the metal used in the manufacture of the exchanger. However, to achieve the same heat-carrying capacity, a larger volume of methanol solution than calcium chloride solution had to be circulated, for the ratio of specific heats of the two solutions is 1:1.8. The larger volume, in turn, required larger pumping capacity to overcome the flow losses in the pipes.

Heat extraction varies greatly with the diameter of the frozen column surrounding the freezing pipe. Thus, at start up (when the ground would be unfrozen), the heat available for extraction by the refrigerating machinery

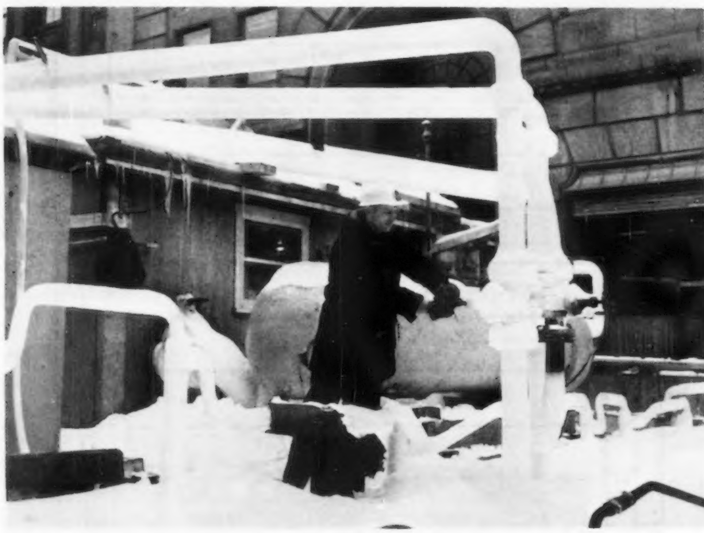

FIG. 2.—OPERATING VALVES AT CROSSOVER HELP BLOW OUT ENTRAPPED AIR.

would be greater than it could handle if it were designed for the working range that would prevail after a few days from starting up. During the first few days, a column of frozen ground would form around the freezing pipes slowing down the rate of heat transfer.

By splitting up the pipe field into sections and opening them to freezing, section by section, load could be applied gradually and kept within the installed capacity of the refrigerating machinery. Therefore, the rectangular pattern of the pipe field was divided in half (Fig. 3). making two squares; inside of the squares lines of pipes divided the entire working area into smaller rectangles, whose long side faced in the direction of the advancing excavation (Fig. 4). By a system of valves, the set of freezing pipes was divided up into individual lines that could be opened or closed without disturbing the overall circulation of the system. By this means, as many pipes as the plant could satisfy were opened to circulation.

Consideration of heat flow indicated that a curtain should be formed on the preimeter of each square, through which no further heat could enter from outside (Fig. 5). Only after this were the pipes within the block opened in turn to form a frozen mass of material ahead of the advancing excavation.

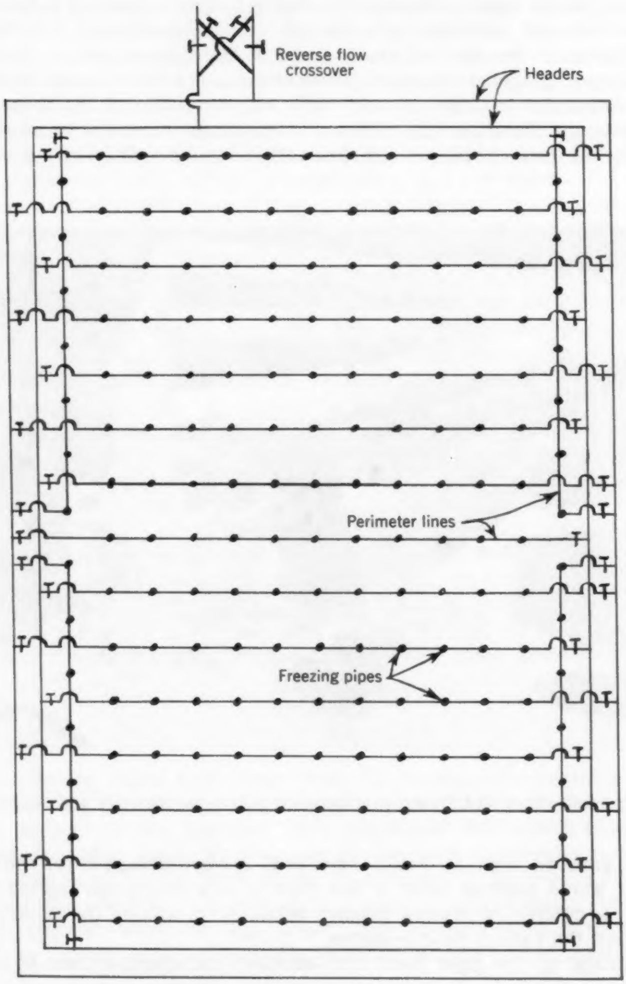

FIG. 3.—DIAGRAMMATIC OUTLINE OF FREEZING-PIPE FIELD.

Actually, it was found that the freezing plant could handle the perimeter pipes and about six rows ahead of the advancing excavation (Fig. 6). As the excavation reached each row of freezing pipes they were shut off, emptied of their methanol solution, and used as supports for scaffolding at the working face.

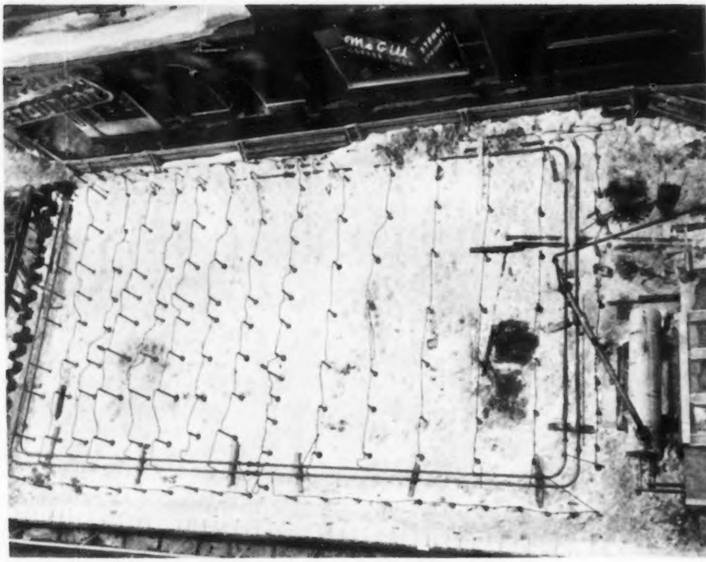

FIG. 4.—FREEZING PIPE FIELD JUST PRIOR TO STARTING REFRIGERATION.

FIG. 5.—PERIMETER PIPES OPENED TO CIRCULATION FREEZE A CURTAIN AROUND THE OPERATING AREA FIRST.

FIG. 6.—PERIMETER PIPES MAINTAINING FROZEN CURTAIN WHILE OTHER LINES FREEZE AHEAD OF THE ADVANCING EXCAVATION; UNFROSTED PIPES HAVE BEEN CUT OUT OF CIRCULATION AFTER SERVING THEIR PURPOSE.

FIG. 7.—INSIDE THE FROZEN EXCAVATION. THE TRACKS ARE LAID DIRECTLY ON THE OLD ARCHES. FROSTING CAN BE SEEN ON THE FREEZING PIPES AND THE FROZEN SILT.

The frozen material excavated much as a weak rock or poor concrete, by drilling and blasting, using, however, a higher powder factor than for either one. A property of toughness was noticed that reacted better to a slow-acting powder than a quick one.

Circumstances permitted the excavation to proceed to the new arch outline without disturbing the old arches. By using the old arches as a working platform, the construction operations did not interfere with the trains running just below (Fig. 7).

Tunnelling proceeded on a production basis of about 1½ ft per day. The day shift loaded and fired the explosives and removed the broken up frozen silt, leaving a clean face for the afternoon shift to drill into (Fig. 8). When drilling, it was found that the heat generated by the bit thawed out the silt enough to form a mud that would clog the hole. Consequently, the driller was obliged to drill in short bursts only, yet maintaining air flow to cool the steel and blow the broken material out of the hole.

Mucking from the face was accomplished by loading a specially built skip by hand and winching it to the portal, from which it was hauled out and dumped into trucks at street level. This skip had to be low enough to pass under the trusses supporting the sewer described previously, and over the old arches, a clearance of only 30 in. The skip was mounted on a 7 ft gauge light railway track supported by the old concrete arches that formed the working platform for operations. Working space was severely limited, for the maxiumum headroom was only 10 ft at the awkwardly shaped face, decreasing progressively as each layer of the new tunnel section was built. This lack of space required more handwork than desirable.

About a day behind the advancing excavation, the erection of an arch of steel liner plates and ribs followed. A good packing, from 3 in. to 10 in. thick, was ensured between the steel and silt by placing grout pneumatically by the Gunite method. The fact that the grout was placed against a face at a temperature of about 28 F did not matter, particularly, since it was used purely as a packing to transmit loads to the steel arch. About a week behind, the erection of the steel arch followed the construction of the permanent reinforced concrete arch. Here again, the concrete was placed by the Gunite method, using forms made from a heavy gauge of expanded metal, and held in place by bolts welded to the steel arch ribs. In this case, however, care was taken to ensure that the concrete was placed at a temperature of not less than 40° F. It was found that the grout placed earlier acted as sufficient insulation to prevent undue heat loss from the fresh concrete to the silt (Fig. 9).

After the permanent concrete arch was completed, together with its end bulkheads, the old twin arch system was demolished by the night shift, who attended to the demolition of the old concrete arches by drilling and blasting the blocks down onto the tracks that had been protected by heavy timbers. The broken concrete was then removed by a front-end loader working from inside the tunnel and a crane working from street level. This demolition operation had to be done particularly well, since all work for the shift had to be completed during the few hours available at night so that the undemolished arches could be made safe for the first train service of the day.

The open-cut section was not completed as an arch, but was roofed over with prestressed concrete beams, high enough to satisfy railway requirements for a tunnel, yet low enough to act as a bridge over the tunnel for the possible future Montreal subway, running below street level.

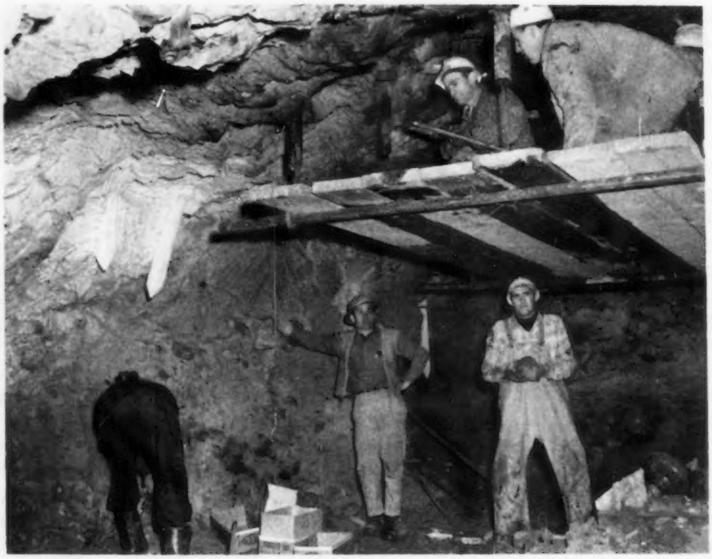

FIG. 8.—AT WORKING FACE, FREEZING PIPES WERE EXPOSED WHILE THOSE NOT IN USE WERE USED FOR SCAFFOLDING SUPPORTS.

FIG. 9.—VIEW OF RAILWAY TUNNEL SHOWS THE CONCRETE BEAMS IN PLACE, THE OLD ARCHES, AND ABOVE, THE NEW TUNNEL ARCH WITH REINFORCING READY TO BE CONCRETED.

It was computed that the amount of expansion due to the 10% increase in volume taking place in the silt layer during freezing, would be accounted for by an upward heave in the street of about 5-in. This was found to be the case. The resulting cracks badly broke up the street payment, and this could not be repaved satisfactorily until full thawing had taken place. To be conservative, this work was not done until a full summer had passed.

It was at first anticipated that the freezing operations might take 6 weeks, but in fact, the first excavation at the face of the open cut began within 3 weeks of starting up the refrigeration plant. At the time, it was thought that the low ambient temperatures of about zero had increased the rate of freezing at this face, but thermometers in test holes confirmed that 3 to 4 weeks was the time taken to freeze the ground solid from its original normal temperature of about 48°F.

CONCLUSIONS

The system of freezing to stabilize soils is not new, since it has been used before for shaft sinking through permanent water-bearing rock as well as for other isolated projects. However, in the case of the tunnel alterations previously described, there was little experience available.

The steel arch used might possibly be left out, in a similar future job, if it could be established that there was sufficient cover and sufficient water available in the soil above the excavation to allow a frozen arch to develop. This could be checked structurally and frozen strong enough to withstand the design loads. In the case of this tunnel, however, the water saturated layer was not thick enough over the new tunnel roof to permit the making of a reliable design.

It should be quite possible to pour the new concrete up against the frozen material, since the chemical heat developed from a thick enough section of fresh concrete would be sufficient to overcome the danger of its freezing. In addition, the conductivity of the silt is low enough to hinder the rapid flow of heat away from the fresh concrete.

A total of 2,000 cu yd of silt and clay was frozen in a total time of 30 days. Although cold-weather conditions prevailed at the time, it is doubtful whether temperature conditions would have any noticeable effect on the actual freezing operation. However, prevailing temperature might have a serious effect on the methods used for the excavation of the frozen materials and for the protection of frozen faces exposed by such excavations.

Little trouble was experienced in controlling the freezing system and expelling trapped air from the pipes by using the blow-off cocks and the reverse flow cross-over. Indeed, the degree of control and immunity from weather changes enabled the job to proceed smoothly and tidily.

At the time of construction, there was a feeling of confidence that the outcome of the freezing operation would be satisfactory. In retrospect, it can be said that the freezing method to reconstruct the tunnel was entirely successful, that no unexpected difficulties occurred, and that the stability of the buildings was in no way interfered with.

Under circumstances similar to those eccountered in this project, and in which the customary methods cannot be successfully used, freezing is a perfectly predictable and satisfactory method for stabilizing semi-liquid soils. It permits clean, dry excavation with no danger of loss of material from the

surrounding areas. The method is too expensive for general use and should be used only when cheaper methods will not provide the required degree of safety. However, when used, it works well as a "local anaesthetic."

ACKNOWLEDGMENTS

Acknowledgments for assistance in preparing this paper are due R. O. Stewart, Consulting Engineer of the Canadian National Railways, and D. F. Murphy, Chief Structural Engineer of the Foundation Engineering Corporation of Canada Limited.

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All photographs are by the courtesy of The Canadian National Railways.

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STATISTICAL AND ECONOMIC ANALYSIS OF A BIDDING TREND

By Marvin Gates,[1] M. ASCE

SYNOPSIS

Almost four hundred results of bids for heavy construction contracts are analyzed for correlation between the size of the contract and the difference between the low and second bids.

The results of this analysis are applied to four diversified areas of competitive bidding. Namely, another criteria for ascertaining the probable existence of a gross mistake in a bid is offered; a suggested maximum amount of bid guarantee is treated, the practice of using "going" prices is investigated, and three methods for increasing profits, based on probability factors, are described.

INTRODUCTION

Recently, a contractor bid 30 million dollars to do a heavy construction job. The second bidder asked 40 million dollars to do the same work. At about the same time, on another job, two other contractors submitted identical bids of $1,400,000. Much comment and speculation continues concerning both incidents. One question remaining unanswered is the odds against such occurrences. Quite understandably, if the answer to this question can be found then it may be possible to control the amount of money "left on the table." This latter term is synonymous with the term "spread." Both expressions are interchangeably used throughout this paper to mean the difference between the low and second bids.

Note.—Discussion open until April 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Construction Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. CO 3, November, 1960.

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The ability to predict the probable spread gives the engineer and contractor a new and important tool, equally as important, is the continuing emergence of bidding practice as a scientific method.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically, for convenience of reference in the Appendix.

ACCUMULATION AND SELECTION OF DATA

The results of competitive bids in the construction industry are usually published by the trade and engineering periodicals serving the geographical location of the project. In the New England States, the weekly publication, NERBA, published by the New England Road Builders Association, carries the complete results of highway projects bid in that area. In addition, many states mimeograph and circulate the results of each bid letting to all interested parties, including all contractors prequalified to bid in the state, equipment dealers, material vendors, and engineers. Fig. 1 is a typical release by the Connecticut State Highway Department; it is their practice to publish the complete bid of each of the three lowest bidders; some states publish the complete bid of all bidders.

For the purpose at hand, the writer has drawn from his file of such publications of the bid results published by the State of Connecticut from 1957 to 1959, and the states of New Hampshire and Vermont from 1958 to 1959. Although it is probable that these files are not complete, they do reflect the bidding pattern for these New England States.

From the accumulation of data, certain contracts were rejected as not being representative of the usual highway construction projects encountered. The rejected material included:

1. Specialty contracts such as highway illumination, joint resealing, landscaping, demolition, dredging, and mechanical work.
2. Alternate bids to the base bid.
3. Contracts having less than ten unit items of work. These too were arbitrarily considered to be of a specialty contract nature; albeit the specialty work was of a heavy construction nature.

The remaining data included bids received for the construction of highways, bridges, and drainage. Where specialty work, such as fencing, was an incidential part of these contracts, they were included. The 381 bids used in this analysis represent about $200,000,000 worth of construction contracts.

TABULATION OF DATA

The range of the dollar values of the contracts under discussion varied from a low of $1,700 to a high of $8,025,000. For ease of handling, the 381 observations in this great range, it was desirable to divide the data into about ten groups. Arbitrarily the progression $1000(2^n)$ was used as the basis for forming these groups. Table 1 shows the division of the groups.

The low bid, expressed as a percentage of the second bid, was computed and then rounded to the nearest whole percentage point for each of the contracts,

Number of Bidders: 11
Date of Letting:
February 8, 1960

TOWN OF WESTPORT
Roads in Sherwood Island State Park
Bituminous Concrete
Length 2,830.00 Feet, Width Varies, Depth $2\frac{1}{2}$ Inches

State Park & Forest Commission
and State Agency Funds
Proj. No. S-3119

Items	Unit	Quantity	D'Addario Services Price	D'Addario Services Amount	The DeFonce Const. Co., Inc. Price	The DeFonce Const. Co., Inc. Amount	De Leo Bros. Inc. Price	De Leo Bros. Inc. Amount
Earth Excavation	C.Y.	3,981	.50	1,990.50	.60	2,388.60	.40	1,592.40
Removal of Existing Masonry	C.Y.	1	10.00	10.00	15.00	15.00	20.00	20.00
Trench Excavation 0'-6' Deep	C.Y.	890	1.50	1,335.00	2.00	1,780.00	1.50	1,335.00
Trench Excavation 0'-10' Deep	C.Y.	705	2.00	1,410.00	3.50	2,467.50	1.50	1,057.50
Ditch Excavation	C.Y.	520	2.00	1,040.00	2.00	1,040.00	.60	312.00
Borrow	C.Y.	9,160	1.00	9,160.00	.95	8,702.00	1.25	11,450.00
15" Reinforced Concrete Pipe	L.F.	248	3.50	868.00	4.00	992.00	4.00	992.00
18" Reinforced Concrete Pipe	L.F.	28	4.00	112.00	4.50	126.00	6.00	168.00
15" A.C.C.M. Pipe	L.F.	144	4.25	612.00	5.00	720.00	6.00	864.00
15" A.C.C.M. Pipe (Asbestos Bonded)	L.F.	356	4.75	1,691.00	6.00	2,136.00	6.00	2,136.00
18" A.C.C.M. Pipe (Asbestos Bonded)	L.F.	934	6.25	5,837.50	6.75	6,304.50	6.00	5,604.00
24" A.C.C.M. Pipe (Asbestos Bonded)	L.F.	100	8.00	800.00	8.50	850.00	9.00	900.00
Underdrain with 6" Perforated A.C.C.M. Pipe	L.F.	950	2.75	2,612.50	3.00	2,850.00	3.00	2,850.00
Type "C-L" Catch Basin	Each	10	210.00	2,100.00	225.00	2,250.00	225.00	2,250.00
Class "A" Concrete	C.Y.	10	65.00	650.00	60.00	600.00	65.00	650.00
Subbase	C.Y.	4,850	2.00	9,700.00	1.50	7,275.00	2.00	9,700.00
Formation of Subgrade	S.Y.	17,426	.15	2,613.90	.20	3,485.20	.10	1,742.60
Rolled Gravel Base	C.Y.	2,960	2.40	7,104.00	1.50	4,440.00	2.25	6,660.00
Bituminous Concrete Binder Course	Ton	1,253	9.50	11,903.50	8.75	10,963.75	9.00	11,277.00
Hot Asphalt Concrete	Ton	1,281	9.50	12,169.50	8.75	11,208.75	9.00	11,529.00
Concrete Park Curbing	L.F.	1,960	2.50	4,900.00	2.25	4,410.00	2.00	3,920.00
Two-Cable Guide Railing	L.F.	370	1.65	610.50	1.75	647.50	1.75	647.50
Compensating Type Anchorage	Each	2	40.00	80.00	40.00	80.00	40.00	80.00
Single Post	Each	18	7.50	135.00	7.00	126.00	8.00	144.00
Slope Paving	S.Y.	4	15.00	60.00	15.00	60.00	20.00	80.00
Furnishing and Placing Loam	S.Y.	41,680	.35	14,588.00	.60	25,008.00	.65	27,092.00
Placing Loam	S.Y.	7,920	.25	1,980.00	.20	1,584.00	.30	2,376.00
				$96,072.90		$102,509.80		$107,429.00

Low Bidder - D'Addario Services - $96,072.90

FIG. 1.—OFFICIAL RELEASE OF THE RESULTS OF A BID LETTING BY THE CONNECTICUT STATE HIGHWAY DEPARTMENT.

according to the following formula:

$$P = \frac{\text{low bid}}{\text{second bid}} \times 100\% \qquad (1)$$

The results were tabulated in cells according to the value of P and the size of the contract (see Table 2). Values of P less than 80% were grouped together. In the analysis, they are regarded as being equal to 80%. Actually, only three items fell into this group, and none were as low as 75%.

TABLE 1.—GROUP DIVISION

n (1)	1,000 (2^n) (2)	Contract group, in dollars (3)
0	1,000	
1	2,000	1,000 — 5,000
2	4,000	
		5,000 — 10,000
3	8,000	
		10,000 — 15,000
4	16,000	
		15,000 — 25,000
5	32,000	
		25,000 — 50,000
6	64,000	
		50,000 — 125,000
7	128,000	
		125,000 — 250,000
8	256,000	
		250,000 — 500,000
9	512,000	
		500,000 — 1,000,000
10	1,024,000	
		1,000,000 — 2,000,000
11	2,045,000	
		2,000,000 — 4,000,000
12	4,096,000	
		4,000,000 — 8,000,000
13	8,192,000	

No distinction between the data obtained from the various states was made. Approximately 30% of the data was contributed by New Hampshire, 20% by Vermont and the remaining 50% by Connecticut.

ANALYSIS OF DATA

From Table 2, it can be seen that there is a relatively small amount of data in the first three and the last two groups. For ease of handling as well as maintaining a representative number of samples within each group, the five aforementioned groups were consolidated into two, namely:

$1,000 - $15,000; $2,000,000 - $8,000,000

TABLE 2.—DISTRIBUTION OF CONTRACTS BY SIZE AND PERCENTAGE OF SPREAD FOR CONNECTICUT, VERMONT, AND NEW HAMPSHIRE, 1957-1959[a]

Low bid, in	P = (Low bid + Second bid) x 100%																					
$1,000,000	100	99	98	97	96	95	94	93	92	91	90	89	88	87	86	85	84	83	82	81	80	<80	TOTALS
1 - 5	0	0	0	0	0	1	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	3
5 - 10	0	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
10 - 15	0	0	3	1	0	2	1	1	3	1	0	1	0	0	0	1	1	0	2	0	0	0	17
15 - 25	0	4	2	1	3	3	3	2	4	4	0	0	1	1	1	2	0	0	1	1	0	1	34
25 - 50	0	9	3	7	5	4	2	1	3	0	0	0	1	3	2	0	0	0	1	0	1	1	43
50 - 125	7	6	9	7	6	6	4	6	5	3	3	2	1	0	1	0	1	0	0	0	1	1	69
125 - 250	8	7	10	9	10	5	2	1	2	0	4	1	1	1	1	0	1	0	0	0	0	0	63
250 - 500	8	12	10	9	3	8	2	0	2	2	1	1	0	0	0	0	0	0	0	0	0	0	58
500 - 1,000	6	8	10	5	4	2	2	0	2	1	0	0	0	0	0	0	0	0	0	0	0	0	40
1,000 - 2,000	4	10	4	7	2	2	2	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	34
2,000 - 4,000	0	3	3	2	1	1	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	12
4,000 - 8,000	2	0	0	3	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6
TOTALS	35	59	54	51	35	34	20	15	22	11	10	5	4	5	5	3	3	0	4	1	2	3	381

[a] Official releases from the highway departments of Conn. (1957-1959), Vt. (1958-1959) and N. H. (1958-1959).

The midpoint, C, that represents the average worth of the contracts within the respective group was taken as the geometric mean of the limits of the group. The geometric mean, G_m, is defined as:

$$G_m = \sqrt[n]{a_1 \times a_2 \times a_3 \ldots . a_n} \quad \ldots \ldots \ldots \ldots (2)$$

in which a is a single observation and n is the total number of observations.

The midpoints of the two consolidated groups were taken as the weighted geometric mean of the limits of the original groups comprising them.

The several values of P were similarly averaged so as to yield the geometric mean, $\overline{P}$, for each of the nine groups. The geometric rather than the arithmetic mean was used because of the more typical values obtained when the data are related exponentially.

However, it will be remembered that P and $\overline{P}$ are the ratios of the low bid to the second bid. For the purpose at hand, it was expedient to express the percentage spread as the ratio of the difference between the second and low bid, ΔB, to the low bid, or:

$$P' = \frac{\Delta B}{\text{low bid}} \times 100\% \quad \ldots \ldots \ldots \ldots (3)$$

Eq. 3 was equated to Eq. 1, thereby eliminating the term, ΔB. Similarly:

$$\overline{P}' = 100\left(\frac{100}{\overline{P}} - 1\right) \quad \ldots \ldots \ldots \ldots (4)$$

in which $\overline{P}'$ is the average (geometric mean) spread expressed in terms of percent of the low bid.

The values of $\overline{P}'$ for each of the nine groups are shown as plotted points on Fig. 2. The points lie on a reasonably straight line indicating that a fully logarithmic correlation between $\overline{P}'$ and C exists. By the Method of Least Squares, the best fitting curve, straight line on log-log paper, was found. Table 3 shows the computations used to determine Eq. 5,

$$p = 108\ C^{-0.266} \quad \ldots \ldots \ldots \ldots (5)$$

in which C is the low bid and p is the computed percentage spread.

Expressed in dollars, the average spread, ΔB_{av}, may be stated by multiplying Eq. 5 by C; that yields:

$$\Delta B_{av} = 1.08\ C^{0.734} \quad \ldots \ldots \ldots \ldots (6)$$

Eqs. 5 and 6 are plotted in Fig. 2.

The computed percent spread, p, is compared with $\overline{P}'$ in Table 4.

It appears that the value of $\overline{P}'$ for the group, 2 million to 8 million, is not consistent with the other values (see Fig. 2). Hence, this value was not included in the computations used to determine Eqs. 5 and 6. However, it can be logically deduced that these equations cannot be extrapolated to near infinite values of C. For example, extrapolation of the other plotted points will yield a value of $\overline{P}'$ equal to almost zero as C approaches infinity. Then, for every large value of ΔB, there must exist a corresponding value approximately equal to $-\Delta B$ in order to produce an average value of near zero. But a negative value of ΔB means that the second bidder is low. This, of course, is absurd. Therefore, lacking further data, it is logical to assume that the extrapolated values will, at some point, tend to be assymptotic to the abscissa. By inspection, from Fig. 2, it may be assumed that for values of C greater than 1.5 million dollars

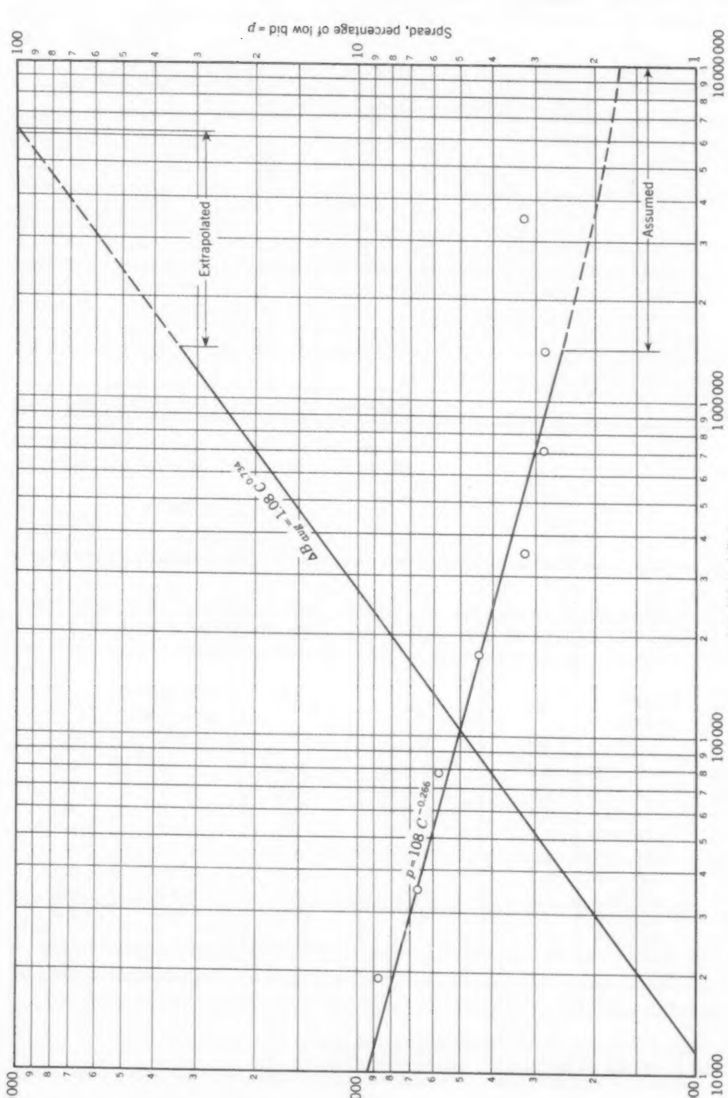

FIG. 2.—LINES OF REGRESSION FOR PERCENTAGE AND EQUIVALENT DOLLAR SPREAD OF CONTRACTS LET BY CONNECTICUT, VERMONT AND NEW HAMPSHIRE, 1957-1959

there is a reduction in the rate of change of $\overline{P}'$. Eqs. 5 and 6, therefore, are true for $10^4 \leqq C \leqq 1.5 \times 10^6$. However, it can be shown that the application of extrapolated values from Eq. 6 will yield conservative results when C exceeds 1.5×10^6.

TABLE 3.—CALCULATIONS OF CONSTANTS FOR LINES OF REGRESSION BY THE METHOD OF LEAST SQUARES

Low bid, in dollars (1)	C (mid-point) (2)	$\overline{P}$ (3)	$\overline{P'}$ (4)	log $\overline{P'}$ (5)	log C (6)	(log C)^2 (7)	log $\overline{P'}$ log C (8)
1,000 – 15,000	9,239	91.904	8.809	0.944927	3.965645	15.72634	3.747245
15,000 - 25,000	19,365	92.083	8.598	0.934397	4.287015	18.37850	4.005740
25,000 - 50,000	35,355	93.812	6.596	0.819281	4.548455	20.68844	3.726463
50,000 - 125,000	79,057	94.588	5.722	0.757548	4.897940	23.98982	3.710425
125,000 - 250,000	176,780	95.798	4.386	0.642069	5.247425	27.53547	3.369209
250,000 - 500,000	353,550	96.893	3.207	0.506099	5.548455	30.78535	2.808068
500,000 - 1,000,000	707,110	97.323	2.751	0.439491	5.849485	34.21647	2.570796
1,000,000 - 2,000,000	1,414,200	97.299	2.776	0.443419	6.150515	37.82883	2.727255
2,000,000 - 8,000,000	3,563,600	96.856	3.246	. . .	. . .	. . .	. . .
TOTALS				5.48723	40.49494	209.14922	26.66520

$$5.48723 = 8.00000 \log a + 40.49494\, b$$
$$26.66520 = 40.49494 \log a + 209.14922\, b$$

$$b = -0.26634$$
$$a = 108.16$$

TABLE 4.—COMPARISON BETWEEN COMPUTED AND OBSERVED SPREADS IN TERMS OF PERCENTAGE AND DOLLARS: AND PROBABLE ERROR[a]

Low bid, in \$1,000,000 (1)	C (mid-point) (2)	$\overline{P'}$ (3)	p[b] (4)	p-$\overline{P'}$ (5)	Difference, in dollars (6)	Error[d] (7)
1 - 15	9,239	8.8	9.3	0.5	46	3.8
15 - 25	19,365	8.6	7.8	-0.8	-154	-7.3
25 - 50	35,355	6.6	6.6	0.0	0	0
50 - 125	79,057	5.7	5.4	-0.3	-237	4.0
125 - 250	176,780	4.4	4.4	0.0	0	0
250 - 500	353,550	3.2	3.6	0.4	1414	8.3
500 - 1,000	707,110	2.8	3.0	0.2	1414	5.0
1,000 - 2,000	1,414,200	2.8	2.5	-0.3	-4242	-8.6
2,000 - 8,000	3,563,600	3.2	. . .	. . .	. . .	. . .
Algebraic total						-2.8
Probable error = - 2.8/8 = - 0.35						

[a] Source: Fig. 2 and Fig. 3. [b] From Fig. 2 and Eq. 5. [c] C (p - P')/100.
[d] Expressed in terms of percentile rating (see Fig. 3).

The assymptotic tendency exhibited on the other end of the curve, $\overline{P}'$ - 8.8. may not be representative and may be due to the lack of a representative cross section of results in the first three groups (see Table 2).

The possibility that the plotted points tend to a Gompertz, or other higher relationship, does not appear to deserve consideration in view of the limited quantity of data in the extreme groups on both ends of Table 2.

Although Eq. 6 gives the average spread, it is also important to know the probability of deviating from this value by some predetermined amount. Ordinary elementary statistical practice could not be successfully applied here because of the skewness of the data as evidenced by the assymetrical distribution (see Table 2).

All of the bid results were plotted on a logarithmic scale, ΔB versus C, (scatter diagram). Then Eq. 6 was plotted, as well as other members of the same curve family, namely, $\Delta B = 1.08\ a\ C^{0.734}$, in which a is a dimensionless parameter ranging in value from 0.02 to 7.0. The cumulative number of results (plotted points), lying under each of the curves was tallied (an ogive). Only the results from C = \$10,000 to C = \$1,500,000 were used in this portion of the analysis.

The result of the tally is shown in Table 5. The cumulative total of 368 includes 20 additional results that became available at the time the ogive was

TABLE 5.—OGIVE OF ΔB LESS THAN 1.08 a $C^{0.734}$

a (1)	No. of contracts (2)	Cumulative total (3)	Cumulative total divided by 368 (4)	1.08 a (5)
0.02	2	2	0.01	0.02
0.033	6	8	0.02	0.04
0.05	6	14	0.04	0.05
0.10	10	24	0.07	0.11
0.20	29	53	0.14	0.22
0.25	14	67	0.18	0.27
0.33	32	99	0.27	0.36
0.50	35	134	0.36	0.54
0.67	30	164	0.45	0.72
1	81	245	0.67	1.08
1.5	50	295	0.80	1.62
2	29	324	0.88	2.16
3	32	356	0.97	3.24
4	8	364	0.99	4.32
5	0	364	0.99	5.40
6	3	367	1.00	6.48
7	1	368	1.00	7.56

being constructed. They are not included in the preceding portions of this analysis.

It can be seen that when $a \leqq 1$, the coefficient, 1.08 a, is equal to an average of 1.6 times the ratio of results (bids) that will be equal to or less than ΔB.

Therefore, the equation of the median trend line is, approximately:

$$\Delta B_{median} = 0.80\ C^{0.734} \quad \ldots\ldots\ldots\ldots\ldots (7)$$

In other words, for a group of contracts, each bid at C dollars, half of the time the spread will be less than ΔB from Eq. 7 and half of the time greater.

Consequently, Eq. 6 may be expanded and transposed to yield the following general relationships between the bid price, C, and any given spread, ΔB:

$$\frac{67\ \Delta B}{1.80\ C^{0.734}} = \text{constant} \quad \ldots\ldots\ldots\ldots\ldots (8)$$

in which the constant is equal to the number of chances out of 100 of a spread equal to or less than ΔB occurring when ΔB is not greater than $1.08\ C^{0.734}$.

Fig. 3 graphically illustrates the solution to the average spread as well as the percentage of results equal to or less than a given spread that may be expected. The probable error of these results is shown in Table 4 as about -0.35. That is to say that the mean is not exactly at the 67 percentile but rather at about the 66.65 percentile. This error is proportional to the percentile rating, and, consequently, is negligible in our study.

Eqs. 6 and 7 are shown as the sixty-seventh and fiftieth percentiles in Fig. 3.

Before concluding this analysis, a word of explanation regarding the term probable error. This value is based on the theorem that the algebraic total of the deviations about the mean is equal to zero. It is not the statistical measure known as the standard error of estimate that is the root-mean-square. This latter measure is not applicable when the data deviates greatly from the normal (Gaussian) distribution. In the case at hand, this measure indicates that almost 100% (99.7%) of the values of ΔB will fall between the 50 and 83 percentiles. This conclusion, however, contradicts the facts. There is also some question as to the use of this measure when the data are related logarithmically, because the logarithm of zero is minus infinity.

Expressing the error in terms of percentile rating at the mean (67) level rather than in terms of percentage or dollars gives a truer picture of the deviations. Expressed in terms of percentage, the error is exaggerated for large values of C. While expressed in terms of dollars, the error is exaggerated for small values of C.

The odds against an occurrence at or greater than the one hundreth percentile level is more than 368 to 1. This is comparable to a deviation from the arithmetic mean of a normal distribution greater than three standard errors. Hence, this area of Fig. 3 is termed statistically impossible.

Fig. 3 then, graphically illustrates the deviations about the mean expressed in percentile form as well as laying the foundation for the next portion of this paper, the practical applications of this analysis.

OTHER CONSIDERATIONS

Every contractor's economic status is unique. Many times the control of available raw materials, such as borrow, will set one contractor distinctly apart from his competitors, when bidding for work. Then again, one contractor may have recently completed a similar project and has at his disposal detailed cost records. Another contractor may be determined to be low bidder at any cost, in order to keep his organization intact. And yet another may have discovered an error in the bid plans and has unbalanced his bid. The list of factors both tangible and intangible, objective and subjective, affecting the final bid price is endless. How then, can the methods described be generally applied?

It has been a long recognized fact that there is a stability of mass data, although the mass is comprised of erratic individual cases. This fundamental precept underlies the basis of actuarial science and is the foundation of the insurance industry. The reader is referred to any standard text on statistics for a detailed discussion of this and other statistical topics.[2]

[2] "Elementary Statistical Methods," by Wm. A. Neiswanger, The Macmillan Co., New York, 1953.

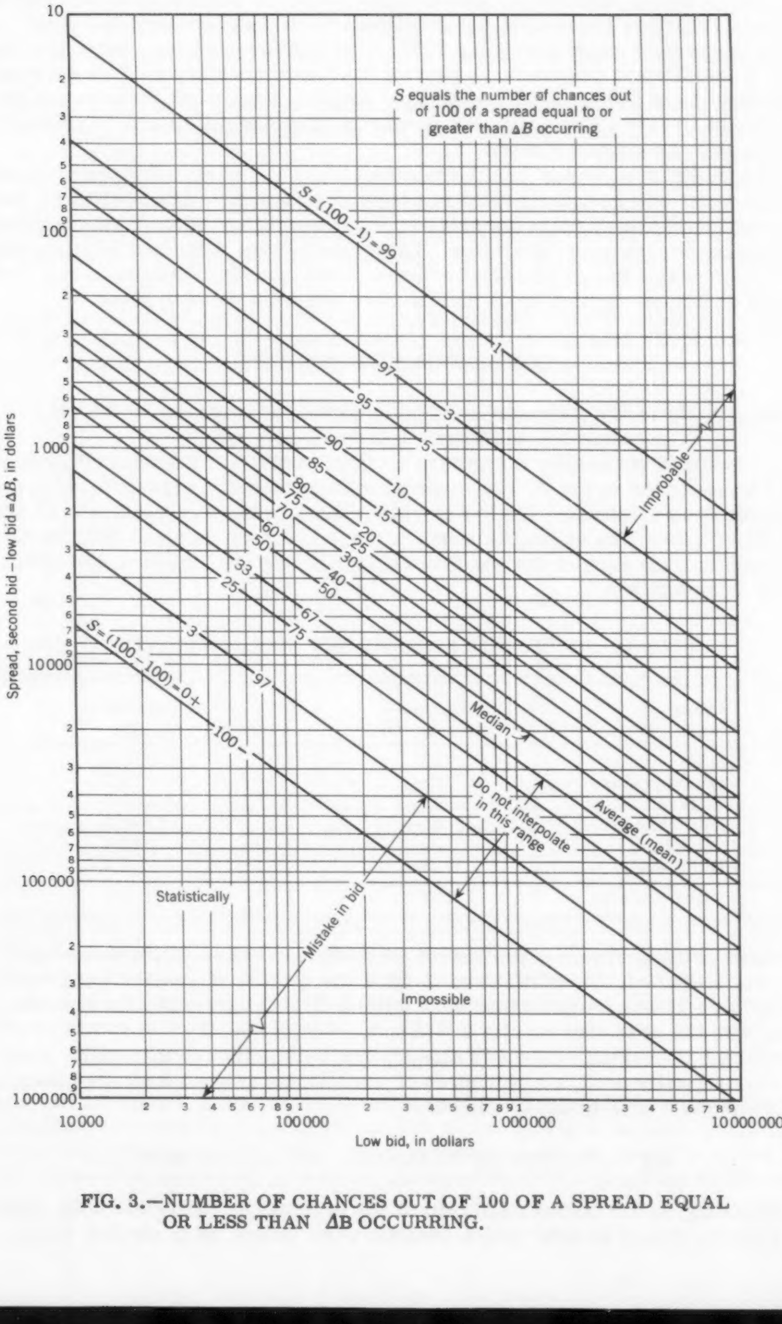

FIG. 3.—NUMBER OF CHANCES OUT OF 100 OF A SPREAD EQUAL OR LESS THAN ΔB OCCURRING.

It is important to note again that the data on which this paper is based originated in three New England States from the year 1957 to 1959. This period was one of generally declining prices in the heavy construction industry. The geographical area had seen a substantial growth in the number and the potential of the contractors because of the accelerated toll road construction program in the immediate years preceding 1957. The highway program, subsequently, was of insufficient magnitude to provide adequate work for an over extended industry. As of Spring, 1960, competition remains keen, work remains scarce, and prices are low. Meanwhile, the number of business failures in this industry continues at an alarming rate.

At present, it is a moot question whether or not the relationships established in this paper will be equally valid in a time of prosperity. The possibility that the relationships may not accurately represent conditions in other geographical areas also deserves consideration. These two factors, time and location, are also major variables in actuarial science. Consequently, the results reported herein should be verified from time to time and from place to place.

CORROBORATIVE RESULTS

In order to verify the accuracy of the analysis, in general, and the reliability of Fig. 3, in particular, additional data was collected.

All itemized bid results reported in the first twenty-six issues of NERBA, in 1960, were plotted on Fig. 3. Non-representative contracts were excluded on the same basis as originally. The low and high contracts were respectively, $3,230 and $3,870,888. The aggregate worth of these contracts is about $60,000,000. Bid results from each of the six New England States are included; the distribution is as follows:

TABLE 6.—BID RESULTS FROM SIX NEW ENGLAND STATES

State (1)	Number (2)	Percentage (3)
Maine	17	12.3
New Hampshire	25	18.1
Vermont	11	8.0
Massachusetts	61	44.2
Connecticut	21	15.2
Rhode Island	3	2.2
Total Contracts	138	100.0

Based on the positions of the plotted points an ogive was constructed, similar to Table 5. Except that in the case at hand the number of results lying on or lower than a given percentile line were tallied. The observed results are compared with the predicted results in Table 7.

Although the stratification and size of this sample differs materially from the one originally used, a high degree of correlation exists. And, the general application of Fig. 3 is substantiated.

APPLICATION OF ANALYSIS: MISTAKE IN BID

Depending on the terms stipulated in the proposal, a contractor may have the right to refuse to enter into a contract even though he is the low bidder,

provided he can prove a mistake in his bid. In certain situations, the contractor may have the right to withdraw his bid without penalty, that is, no forfeiture of the bid guarantee. The theory at law is that when a mistake is so large as to be obvious to the offeree, then the offeree knows that the bid is not the intended offer; however, neither does he know what the intended offer is. Therefore, no meeting of the minds ever occurred, and there is no basis for a contract.

Even though it is apparent that a bid is in error, by comparison with the other bids and the engineers' estimate, there is very often no other tangible evidence of the presence of a mistake. When this is the case, the contractor or his surety may be held bound to the forfeiture provisions in the proposal.

As a result of the analysis it is possible, statistically speaking, to ascertain at least one criteria for establishing the strongest probability that a mistake is manifest in a bid. Statistically speaking, if the chances of an event occurring

TABLE 7.—COMPARISON BETWEEN PREDICTED AND OBSERVED SPREADS IN TERMS OF PERCENTILE GROUPS AND AN EQUAL TO OR LESS THAN OGIVE[a]

Percentile group (1)	Individual group		Cumulative group	
	Predicted (2)	Observed (3)	Predicted (4)	Observed (5)
Less than 1	1	3	1	3
1 - 3	3	4	4	7
3 - 5	3	2	7	9
5 - 10	7	5	14	14
10 - 15	7	11	21	25
15 - 20	7	5	28	30
20 - 25	7	8	35	38
25 - 30	7	8	41	46
30 - 40	14	11	55	57
40 - 50	14	8	69	65
50 - 67	23	22	92	87
67 - 75	11	11	104	98
75 - 97	30	34	134	132
97 - 100	4	6	138	138
More than 100	0	0	138	138

[a] "NERBA," January 2, 1960 to June 25, 1960, inclusive and Fig. 3.

are less than 3 out of 100, then it is an improbability that the event will occur. Therefore, if the difference between the low and second bids, ΔB, exceeds the 97 percentile, the low bid may be viewed as a rare item or a freak.

It is, therefore, suggested that it be considered "prima facie" evidence of the existence of a gross mistake in a bid if:

$$\Delta B \geqq 3.24\, C^{0.734} \quad \text{. . . . . (9)}$$

and the low bidder, if he requests, and if permitted by statute, be permitted to withdraw his bid without penalty. Eq. 9 is shown on Fig. 3 as the 97 percentile. This concession admits to 3% of the low bids as probably being in error, although some of these errors may be attributed to such intangibles as inexperience and ignorance.

A recent article[3] reports that the Corps of Engineers will include, in their future contracts, the following clause relating to bid mistakes:

> "Mistakes in Bids. The bidder hereby waives that portion of any alleged mistake or mistakes in his bid which falls within the following amounts:
>
> If bid is $250,000 or less - 5% of the bid;
>
> If bid is more than $250,000 and less than $500,000 - $12,500 plus 4% of the bid over $250,000;
>
> If bid is $500,000 or more, and less than $1,000,000 - $22,500 plus 3% of the bid over $500,000;
>
> If bid is $1,000,000 or more - $37,500 plus 2% of bid over $1,000,000.
>
> In cases where the allegation of mistakes exceeds the above waived amounts and the request for correction is allowed, such amount will be excluded from the contract price; however, the amount waived as provided herein will not be deducted for the purpose of evaluating bids to determine the low bidder.
>
> The above waiver does not apply to any clerical mistake which is obvious or apparent on the face of the bid including but not limited to (1) mistake in the extension of a unit price or prices; (2) a mistake in totaling the sums of various bid items; (3) obviously misplaced decimal point; or (4) failure to insert the unit price where amount intended can be determined from face of bid.
>
> This clause is not applicable to allegations of mistakes, which if allowed, would result in a reduction in the bid price."

It is interesting to note that the preceding schedule, if plotted on Fig. 3, would fall between the fortieth and seventy-fifth percentiles. In other words, the amount that must be waived is approximately equal to the average spread from Eq. 6. Furthermore, it recognizes the existance of two limits of p, one at $p = 5\%$, and the other, an assymptote at $p = 2\%$ (see Fig. 2). It is probable, however, that the aforementioned schedule was developed for ease of application with some sacrifice of theoretical precision, if we are to assume that the amount to be waived is intended to be related to the mean spread.

Nevertheless, mistakes in bidding are made for reasons other than mathematical or because of oversight. Specifications may be misinterpreted, anticipated sources of materials purported to be acceptable may have been misrepresented, out-of-town contractors may not have been informed of peculiar pertinant local ordinances, and the nature of payment for one or more items of work may be construed (one state classifies as earth excavation the removal of a soil that requires drilling and blasting). It is not possible for a contractor to prove these allegations objectively. Neither can these factors be reduced to monetary terms without inviting skeptical replies. And yet, an unusually large sum of money left on the table is almost universally held as being indicative of an error in the bid.

In cases of extreme difference between the low and second bids, the low bidder should be permitted to withdraw his bid without penalty, since it can now be immediately ascertained if the low bid was conceived in error. On the other hand, if the low bidder has found an unusually economical method of doing a portion of the work, and he has reflected this potential economy in his esti-

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[3] "Roads and Streets," March, 1960, P. 108. Quoting from The Associated Gen. Contractors of Mo.

mate, then he will not ask to have his bid voided. However, this possibility should not be misconstrued and used as justification for taking advantage of an obviously erroneous proposal.

BID SECURITY

As a guarantee that the lowest qualified bidder will enter into a contract with the owner, at the prices bid and under the terms of the proposal, all bidders are required to submit with the bid some form of bid security. As a rule, the bid securities of all but the lowest three bidders are returned immediately after the bids have been checked. After the contract is signed, the remaining securities are returned.

The bid guarantee may be in the form of a certified check, bid bond, or negotiable securities. Some states will accept the bid security in any of these three forms. Other states will not accept a bond. Still others require that the bid security must be in the form of a bond. The form of the bid security is not standard and may even vary between agencies of the same state.

The amount of the bid security varies even more widely than the form. It is customary to specify the amount of the bid security in one of three ways:

1. A percentage of the bid. This usually varies from 5% to 33 1/3%, depending on the policy of the agency.
2. A fixed dollar value. This amount is determined by the agency and is usually predicted on the engineer's estimate of the cost of construction. Many agencies employing this method set a ceiling on the amount of the security. One state uses a sliding scale for jobs costing up to $500,000. For all contracts above this value the bid security is $10,000.
3. A percentage of the bid and a minimum fixed dollar value.

It is not the purpose of the writer to compare the merits of each of the three forms and types of bid securities. Rather, it is intended to apply the previously reported statistical analysis to this aspect of competitive bidding with the aim of determining the maximum justifiable amount of bid security.

If the low bidder is awarded, but refuses to sign the contract, he must forfeit his bid guarantee. If the agency elects to award the contract to the second bidder, the cost of owning the work is increased by an amount equal to the difference between the low and second bids. Since the bid security is not a penalty for refusing to sign the contract, but rather a form of insurance against the loss of the financial advantage of the low bid, the amount of the bid security should be equal to the amount of loss that can be reasonably anticipated. Or, should the agency reject all the bids and readvertise the job, rather than award it to the second low bidder, the probability is that the amount of loss of financial advantage in this instance will not exceed that of the first alternate. That is to say, the low bid of the readvertised proposal will hardly ever exceed the second low bid of the original proposal.

To be protected 97% of the time for the loss of financial advantage, if the low bidder refuses to enter into a contract, the amount of bid security should be equal to the maximum amount that will be left on the table 97% of the time. This amount is shown as the 97th percentile in Fig. 3, and as Eq. 9. This limit is suggested as the maximum limit because it is the boundary of the realm of probability. The rare eases wherein the spread exceeds this limit have been discussed under the heading "Mistake In Bid."

A certain New England State Highway Department requires bid security in the form of a bond in the amount of one third of the bid price. If the low bidder for a job in this state bids \$150,000 and leaves \$50,000 on the table, that is, second bid is \$200,000, he must either enter into a contract and possibly lose \$50,000, or he may forfeit his bid bond, that, too, is for \$50,000. From Fig. 3 it can be seen that this amount of security protects this state for an occurrence not likely in one time out of four hundred. Moreover, this policy gives the contractor little choice of whether of not to proceed with the work or of forfeiting his bid guarantee. However, according to Fig. 3, the 97th percentile indicates a bid security of from \$20,000 to \$25,000 as being sufficient insurance for all cases within the realm of probability for contracts of this size. Nevertheless, again referring to Fig. 3, a bid security of one third is justified for contracts less than \$10,000.

The amount of the bid guarantee is often a decisive factor in the economic life of a construction company. This singular factor is too often arbitrarily chosen, sometimes to the disadvantage of the owner and also to the disadvantage of the contractor. Logically, the amount of bid security should be based on the intended purpose of this requirement, namely, to reflect the possible loss of advantage of the low bid. But this potential loss is a measurable quantity. Hence the maximum amount of bid security should be a standardized requirement consistent with the facts introduced herein.

It appears that the most equitable arrangement is to require that bid bonds insure the owner for an amount equal to the actual difference between the low and second bids. But, in addition, provide an upper forfeiture limit not in excess of $3\,\Delta B_{av}$. (Note that $3\,\Delta B_{av}$ equals $3.24\,C^{0.734}$ and is also equal to ΔB from Eq. 9). When the bid security is in a form other than a bond, the amount to be posted should not exceed $3\,\Delta B_{av}$. In the event that the low bidder elects not to contract for the work, his guarantee, less the difference between his bid and the second bid, should be refunded to him.

USE OF AVERAGE BID PRICES

The items of work enumerated in a unit price proposal may be classified according to the following categories:

1. Minor
2. Subcontract
3. Major

These divisions are not mutually exclusive, since portions of the work required under a single item may fall into all three categories. For example, a contractor bidding for a job requiring structural concrete may find that the cost of placing the concrete is minor, whereas the expense of forming and stripping is major. Any finishing, such as rubbing and painting, the contractor will "sub out."

In the case of the unique contract representative of the 381 contracts used in this example, the average of each of the unit bid prices may yield a total bid that is in line with the other bids. There is, however, no such contract. The writer does, however, recall a singular job on which he was the low bidder. On this occasion, none of the unit bid prices he submitted were the lowest tendered. This is not a rare situation, but it does emphasize the justification of the use of going prices, in certain situations at least.

From Fig. 3, 67% of all the contracts had a spread equal to or less than the mean spread. If it is assumed that all the unit items of a proposal are equal in dollar value, then two thirds of the items will probably be bid with a spread at or lower than the average, while the remaining one third of the items will be bid with a spread at or higher than the average. And if it is assumed that the magnitude of the spread is indicative of the precision of the estimating, then it may be concluded that one third of the items may be bid by employing average prices, since the probability is that further refinement will not reduce the probable spread. Therefore, the minor items may be considered as the maximum number of different items with an aggregate worth equal to about one third of the total contract price.

The assumptions underlying this conclusion are not always true. For example, it is possible that the low and second bids were both carelessly bid, yet the spread may be very small. Obviously, there is no corrolation between spread and precision in this instance. Also, there is no such contract wherein all the items are equal in aggregate value. Nevertheless, the suggested approach is expedient.

For some small jobs, every item may appear to be insignificant in itself. And, it may be most difficult to cull out minor items. In the case of certain large jobs, one third of the total contract price may include some items that should be estimated in detail. Great care and experience are requisite to using average prices. One of the items on a job bid by this writer was for "Trench Excavation - 0 ft To Over 15 ft Deep." Seven bidders used the going price of eight to ten dollars a cubic yard. Investigation showed that in this particular case the item would have been overpriced at eighty cents a cubic yard.

It is this writer's practice to use a 10% to 20% factor instead of the one third previously mentioned. And then only after determining that the average prices are applied only to items of work under average conditions.

Minor items of work when considered together constitute about one third of the total dollar value of a project. Should every minor item be poorly bid, the total error by comparison will be a small part of the total bid. Because of this and the fact that the use of minor items is often contingent on field circumstances that one difficult to predict, these items may be bid by using the average or going unit price. If there are a large number of such items, about thirty, the probability is that compensating errors, or the low of averages, will act on the total worth of these items to yield a price comparable to the average value of the minor items, all bids considered.

Items of work that entail soliciting a price, complete from others, and to which is added only the loading (allowance for overhead and profit), are nominally termed subcontract items and are included in this category. Unlike the other two categories, there are no computations required to find the cost for doing these items of work. Consequently, the question of average prices does not arise in these cases, rather, it is supplanted by the ability to trade.

All items of work not counted as either minor or subcontract are termed major. Also, all lump sum items that are not subcontracted must be included here because the going price cannot be readily ascertained. These items should always be bid on the basis of a careful cost study. And, in the case of lump sum items, on the basis of an accurate quantity take-off as well.

Properly applied, average unit bid prices may substantially reduce the time (and cost) of preparing a bid, with no appreciable sacrifice of precision. In certain cases, the use of average unit bid prices may also reduce the amount

of money otherwise left on the table. Imprudent use of this tool, however, may result in irreparable harm.

METHOD OF CONSTANT WORK

Through the years, most contractors have come to estimate their chances of success at bidding. A contractor knows that he will be successful say, one time out of ten. And, after about nine unsuccessful bids will feel that "the next one is mine." If a contractor is successful, on an average of p% of the time, and his company needs n number of jobs a year to maintain an optimum work load, then the minumim number of jobs that must be bid, N, is:

$$N = \frac{100\ n}{p} \qquad (10)$$

From Fig. 3 it is a simple matter to find the probability of leaving a given amount on the table. Conversely, if a given amount is added to the completed bid, the probability of losing the job may also be readily evaluated. For example, a contractor has prepared a completed bid, amounting to $100,000, it has not yet been submitted, and unbeknown to him, his bid is the low bid. According to Table 4, the chances are 30 out of 100 that $2,000 or less will be left on the table. If, at the last minute, the contractor decides to add $2,000 to his bid, he eliminates these 30 out of 100 chances of being low bidder. In other words, while a bid of $100,000 is 100% certain of being the low bid, a bid of $102,000 is 100-30 or 70% certain. If this contractor's past record is p = 10% (from Eq. 10), then by adding the $2,000, p is now equal to 70% of 10%, or 7%.

As p decreases, the number of jobs that must be bid in order to maintain the optimum number of contracts, n, increases. Or,

$$N = \frac{100\ n}{p\ S} \qquad (11)$$

in which S is the success factor and is expressed as the number of chances out of 100 of leaving more than a specified amount on the table. This specified amount is the amount added to the bid in each case. An example will illustrate the application of this method.

The optimum number of jobs a year for a certain contractor is five, $400,000 contracts. In the past years, he has had to bid an average of 50 jobs a year in order to maintain this optimum condition, or $p = \left(\frac{5}{50}\right) \times 100\% = 10\%$. The contractor, in an effort to reduce the amount of money he leaves on the table, decides to arbitrarily add $4,000 to each bid in the future. How many jobs must he now bid in order to be successful the optimum number of times, five. From Fig. 3, the probability of leaving $4,000 or less on the table when bidding $400,000 is 20 chances out of 100. Then the success factor, S, is 100 - 20 or 80%. Substituting in Eq. 11:

$$N = \frac{100\ (5)}{(10)\ 80\%} = 62.5$$

The required number of jobs to be bid in 62.5. If the contractor can reasonably expect this number of jobs to be advertised then he is safe in increasing his bids by $4,000. By so doing, his volume of work will remain constant while

increasing his annual profits by 5 x \$4,000 or \$20,000, at the expense of bidding an additional twelve to thirteen jobs a year.

METHOD OF MAXIMUM INCOME

Every dollar added to a completed bid increases the anticipated net profit by a like amount. However, any such increase also reduces the chances of being the low bidder. The optimum condition exists when the probable gain minus the probable loss is a maximum value. The probable gain is equal to the success factor, S, multiplied by the corresponding value of the bid increase, Δb. The probable loss is equal to 100 minus S multiplied by the anticipated profits, p_a, or:

$$(S \Delta b) - p_a(100 - S) = \text{maximum} \quad \ldots \ldots \ldots \ldots \quad (12)$$

This expression may also be stated as:

$$S(\Delta b + p_a) - 100\, p_a = \text{maximum} \quad \ldots \ldots \ldots \ldots \quad (13)$$

But, as previously pointed out, Δb is directly proportional to $(100 - S)$ when $(100 - S)$ is equal to or less than 67. Then if $\Delta B_{(1)}$ represents the spread when $(100 - S)$ equals unity, a direct proportion may be set up from which:

$$\Delta b = (100 - S)\, \Delta B_{(1)} \quad \ldots \ldots \ldots \ldots \quad (14)$$

Now express p_a in terms of $\Delta B_{(1)}$, or:

$$p_a = j\, \Delta B_{(1)} \quad \ldots \ldots \ldots \ldots \quad (15)$$

Substituting Eqs. 14 and 15 in Eq. 13, factoring and simplifying yields:

$$S(100 - S) + j\,(S - 100) = \text{maximum} \quad \ldots \ldots \ldots \ldots \quad (16)$$

By differentiation, the maximum value is obtained when:

$$S = \frac{1}{2}\,(100 + j) \quad \ldots \ldots \ldots \ldots \quad (17)$$

in which j is equal to $p_a/\Delta B_{(1)}$. Values of S greater than 100 are meaningless and should be considered equal to 100.

An extension of the example cited under the heading Method of Constant Work will illustrate the application of this, the Method of Maximum Income.

A contractor has prepared a bid for \$400,000. If he is the successful bidder he anticipates a net profit, in the bank, of \$10,000. How much, if anything, should he add to his bid for maximum income? Then:

$$p_a = \$10{,}000$$

and from Fig. 3:

$$\Delta B_{(1)} = \$200$$

so that:

$$j = 10{,}000/200 = 50$$

Now substituting in Eq. 17:

$$S = \frac{1}{2}\,(100 + 50) = 75$$

and again from Fig. 3:

$$\Delta b = \$5{,}000$$

Therefore, the optimum amount this contractor should add to his bid is \$5,000. By doing this, he increases his anticipated profits by (\$5,000/\$10,000) x 100% or 50%. However, this is done at some relatively small risk. Because, if his original bid of \$400,000 was the low bid, now his chances are reduced to 75 out of 100 that his bid of \$405,000 will still be low.

The contractor should now apply the previously described Method of Constant Work to determine the effect of increasing his bid and corresponding reduction in his probability of being successful on the number of jobs he must bid in order to maintain the optimum work load for his company. In fine, the value of S should be evaluated in terms of both methods discussed.

Table 8 illustrates the probable financial advantages resulting from the application of this method. The example used in the text is also used as the basis for making the comparisons. In the table, two types of revised bids are considered. In the first instance, the number of jobs bid, N, is kept constant with respect to the original case. In the second instance, the number of successful (low) bids, n, is the constant. In either case, the probable financial advantage is significant.

Eqs. 14 and 17 may be combined to yield the general expression for finding the optimum amount to be added to the completed bid, directly:

$$\Delta b = \Delta B_{median} - \frac{p_a}{2} \qquad (18)$$

If Eq. 18 is used, the success factor, S, is found directly from Fig. 3.

In solving the illustrative examples, the longer, step-by-step method is used rather than Eq. 18. This is done only to better acquaint the reader with the mechanics of the solutions.

METHOD OF MAXIMUM PROFIT

This method is but a modification of and corrolary to the Method of Maximum Income. It has the advantage of giving the amount of profit to be added to the raw cost of the bid by a one step computation. However, it lacks the prime facility of determining a value for S. Nevertheless, if work is plentiful and past experience indicates that it is not practical to attempt to estimate the net profit from a bid, then this method will prove to be expedient. In this discussion, the symbol p_a is equal to and is used in place of the symbol Δb, since Δb is herein equal to the amount to be added to the raw cost.

From the Method of Maximum Income, when S equals 100, nothing is added to the bid. In other words, p_a is a maximum value. But when S equals 100, j must equal 100. Then,

$$\frac{p_a}{\Delta B_{(1)}} = 100$$

but,

$$\Delta B_{(1)} = \frac{\Delta B_{av}}{67} \qquad (19)$$

in which $\Delta B_{(1)}$ is the spreadat the first (1st) percentile previously discussed. Substituting in Eq. 17 and simplifying:

$$S = 50 + \frac{33\,p_a}{\Delta B_{av}} \qquad (20)$$

Then for S to equal 100,

$$p_a = 1.5\,\Delta B_{av} \qquad (21)$$

This is the allowance for profit to be added to the raw cost.

A more conservative approach is to add to the raw cost of the bid the average spread, ΔB_{av}, since this value is the average spread that will occur with

TABLE 8.—COMPARISON BETWEEN EARNINGS

Item (1)	Original (2)	Revised; S = 75%: Constant N (3)	Revised; S = 75%: Constant n (4)
Number of jobs bid, N	50	50	66.67
Number of successful bids, n	5	3.75	5
Percentage of time successful, p	10	7.5	7.5
Amount of bid, intended, C	\$ 400,000	\$ 400,000	\$ 400,000
Amount added to bid, Δb	0	\$ 5,000	\$ 5,000
Amount of bid, submitted, $C + \Delta b$	\$ 400,000	\$ 405,000	\$ 405,000
Anticipated profit originally, p_a	\$ 10,000	\$ 10,000	\$ 10,000
Anticipated profit revised, $p_a + \Delta b$	\$ 10,000	\$ 15,000	\$ 15,000
Gross income, $n\,(C + \Delta b)$	\$2,000,000	\$1,518,750	\$2,025,000
Net income, $n\,(p_a + \Delta b)$	\$ 50,000	\$ 56,250	\$ 75,000

contracts containing an allowance for profit. The success factor in this case is, from Eq. 17:

$$S = \frac{1}{2}\left(100 + \frac{\Delta B_{av}}{\Delta B_{(1)}}\right) \qquad (22)$$

but $\Delta B_{av}/\Delta B_{(1)}$ equals 67. Therefore, S equals 83.5. Now, if the bid in the form of the raw cost, plus an allowance for profit equal to ΔB_{av} is 100 percent certain of being the low bid, then by adding an additional 16.5 $\Delta B_{(1)}$ to the bid it becomes 83.5 certain of being low. The total allowance for profit to be added to the raw cost is:

$$p_a = 1.25\,\Delta B_{av} \qquad (23)$$

A still more conservative approach is to say that the contractor is prepared to bid the job at cost. Then from Eq. 17, S equals 50, or,

$$p_a = 0.75\,\Delta B_{av} \qquad (24)$$

Eq. 24 is the fiftieth percentile in Fig. 3.

There are, of course, an infinite number of values for p_a. The three outlined previously are presented only to facilitate a choice.

CONCLUSIONS

It is evident that a rational approach exists where intuition and hunches now prevail. It has been the primary aim of this writer to illustrate the fact that a logical and mathematical solution exists to problems of competitive bidding[4] just as it does for other problems in construction engineering. The specific techniques and suggested applications must be subrogated to this proposition.

The conclusions as such are manifested in the latter portions of the paper under the heading "Applications of Results." In summary, they are:

1. The difference between the low and second bids is related to the amount of the low bid in terms of probabilities.
2. However, additional investigation, using more advanced statistical methods, should be applied. This should be done not only to refine, but also to advance the analysis. For example, the log-probability law may better relate the variables in Table 5 as well as establish a standard error of estimate.
3. Similar provisions, relating to mistakes in bids as used by the Corps of Engineers are justified and should be incorporated in State contracts as well. In addition, some provision should be made in the contract documents to allow an obviously erroneous bid to be withdrawn without penalty. The following clause is suggested:

> "Withdrawal of Erroneous Bid. The bidder may withdraw his bid after bids have been opened, if his bid is low and if the difference between his bid and the second low bid exceeds the following amounts:
>
> If the low bid is $10,000 or less - $3,000; if the bid is more than $10,000 but not more than $50,000 - 30% of the bid; if bid is more than $50,000 but less than $1,000,000 - $15,000 plus 10% of the bid over $50,000; if bid is $1,000,000 or more - $110,000 plus 5% of the bid over $1,000,000.
>
> Withdrawal of the bid shall be without prejudice or penalty."

4. The amount of the bid security should not exceed the amounts listed in the preceding schedule. The forfeiture clause should also provide for reimbursement of the difference between the spread and the amount of bid security posted.
5. Generally speaking, average unit bid prices may be used to price up to one third of the dollar value of the bid without sacrificing precision.
6. The optimum profit to include in a bid is a unique value. This value depends on the probable spread that, in turn, is a function of the amount of the contract.

The application of known disciplines to even so subjective a topic is competitive bidding is logically and mathematically justified. The techniques, though simple, may prove to be a useful tool, for the contractor as a means of

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4 "Aspects of Competitive Bidding: Unbalanced Bidding and A Rational Method for Calculating Bid Prices," 75th Annual Report, Conn. Soc. of Civ. Engrs.

increasing profits, as well as an aid to the owner-engineer to assist him in equating that is reasonable with cautiousness.

ACKNOWLEDGMENT

Valuable, constructive criticisms were made by several of this writer's associates who are in the construction and engineering professions.

APPENDIX A NOTATION

The following symbols are adopted for use in this paper.

a	=	constant;
a	=	value of a sample;
a	=	dimensionless parameter;
b	=	constant;
Δb	=	amount added to the completed bid;
ΔB	=	difference between the low and second bids;
ΔB_{av}	=	average amount left on the table;
ΔB_{median}	=	maximum spread occurring 50% of the time;
$\Delta B_{(1)}$	=	maximum spread occurring 1% of the time;
C	=	amount of the low bid;
C	=	geometric mean of the limits of a contract group;
G_m	=	geometric mean;
j	=	ratio of the anticipated profit to $\Delta B_{(1)}$;
n	=	total number of samples;
n	=	dimensionless parameter;
n	=	number of successful bids in a given time;
N	=	number of bids required to yield n low bids;
p	=	percentage of N equal to n;
p	=	spread expressed as a percentage of the low bid calculated;
p_a	=	anticipated profit;
P	=	low bid expressed a percentage of the second bid;
$\overline{P}$	=	geometric average of P;
P'	=	spread expressed as a percentage of the low bid, observed;
$\overline{P}'$	=	geometric average of P'; and
S	=	percentage of time a spread equal to or greater than ΔB will occur.

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SIMPLE RULES FOR HOT OR COLD WEATHER CONCRETING

By John J. Manning,[1] F. ASCE

SYNOPSIS

A set of rules is provided for concreting in other-than-normal weather conditions. It is pointed out that the problems of hot-weather concreting are as involved as those encountered during cold weather. Adherence to the rules set forth will eliminate most of the major hazards that exist.

INTRODUCTION

In reflecting on the accomplishments of the construction industry and its contribution to ultimate victories in both World Wars, one must be impressed with the fact that at no time was any desired military operation restricted or prevented because of the failure of the construction industry, militarized, if you will, for the war periods, to accomplish whatever construction was necessary and incident to the success of the military operation.

In these days of refinements in the theory of design of structures and the development of new materials for construction, plus the highly competitive conditions which prevail, construction operations have assumed far greater importance than in the past. Similarly, the engineer and technical forces having refined their theories and designs can no longer bask in the relative security of safety factors of four. Modern competition will not permit such extravagance in safety factors.

Note.—Discussion open unitl April 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Construction Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. CO 3, November, 1960.

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It is widely recognized that the finest and most economical design, prepared with the utmost care and accuracy, can be completely vitiated by the improper execution of the design in the construction areas.

No longer can the engineer be content to furnish a set of plans predicated on his masterful design and leave its execution in actual construction to any Tom, Dick, or Harry who, because of his ability to furnish a performance bond, received the construction contract.

It is the obligation of the engineer to make certain of full compliance with the design requirements, either by his own continued contact during the construction period, or by engaging responsible representatives to whom he would delegate control of the construction operations.

The Concrete Industry Board (CIB) of New York, N. Y., for example, came into being because of the threat of certain public agencies to discontinue the use of concrete for structural frames owing to lack of uniformity of the concrete being produced and the questionable strength of the concrete. The objective of this organization is to insure in every way possible the production of quality concrete. Recognizing the contribution of every phase of the professions and the industry in the satisfactory completion of concrete structures, the CIB includes in its membership representatives of each component of the industry. It operates through committees, much as the activities of the ASCE are accomplished. The work of the committees has resulted in the publication of Manuals of Recommended Practice covering various activities in the industry.

One of these is the committee having to do with Concreting Operations During Hot or Cold Weather. It has produced a very concise, brief Manual of "Recommended Practice for Concreting Operations During Hot or Cold Weather." Being local in its origin and membership, reference to concreting in hot or cold weather obviously pertains to the temporate zone and does not cover the great extremes of temperature and weather covered in the articles by Palmer W. Roberts, F. ASCE.[2]

It can readily be recalled when the advent of cold weather meant cessation of concreting operations until the following spring. Little by little this period of inactivity has been shortened. However, the intense activities of the construction industry have compelled us to completely eliminate any slow down or cessation of construction operations because of adverse weather conditions. We must now provide for year-round concreting operations.

It did not occur to us in the earlier days, when we stopped concreting operations during cold weather, that concreting in hot weather could produce almost as many problems and hazards.

As our knowledge of concrete increased and as our research continued, it was clearly demonstrated that hot weather was not the ideal time but, on the contrary, it was necessary to adopt new methods and procedures to offset the effects of extremely hot weather to insure the quality of the concrete. So we seldom speak now of hot or cold weather concreting but combine the two, recognizing that the greater we vary from temperatures of 50 F to 70 F, the greater the potential hazards.

Irrespective of what may be done to offset weather and temperature effects on concrete, to a great extent it will be wasted effort if we are not working with material that is basically proper. This is concrete of the quality necessary to meet design requirements. Unless we can be assured that the basic concrete

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[2] "Adverse Weather," by Palmer W. Roberts, Civil Engineering, June, 1960, p. 35, July, 1960, p. 44.

is of adequate and proven quality we are, one might say, operating in a fool's paradise.

Stop to realize how little can be done to improve the quality of concrete once it has left the mixer! Obversely, realize in how many ways the concrete received from the mixer can be damaged as to its quality! Compare the intricate and minute controls exercised during the manufacturing processes of structural steel with the manufacturing processes of concrete. It is of the utmost importance that the designer, or his duly authorized and qualified representative, carefully follow construction operations.

It should be recalled that all concrete standards and tests are based on so-called normal conditions of climate and temperature. If we are to place concrete under other than these conditions, that is with wide variation in temperature, moisture and wind, it is essential that the variations be recognized and necessary steps taken to prevent the resulting deleterious effects.

It must then be immediately obvious that all the additional procedures required due to variations from normal conditions are predicated on the supposition that the concrete is in all respects a quality concrete, produced with the proper materials, in proper proportions and under proper procedures. Special procedures incident to hot or cold weather can of themselves do nothing to improve the basic quality of the concrete. Without quality concrete with which to work our efforts are useless. It should not be necessary to stress this point by constant repetition but unfortunately it is. Unless we learn to make certain that we are doing first things first, most of our efforts are wasted.

As an indication of the effect of temperatures, it may be noted that in the field an increase of temperature of concrete to 95 F during mixing and hardening may result in the following:

Increase the water requirements by as much as $\frac{1}{2}$ gal per bag of cement
Shorten setting time by one-half
Decrease ultimate strength
Tend to increase both plastic and shrinkage cracking.

While on the other hand a decrease in temperature to 35 F may

Delay setting time by several hours
Delay floor finishing and form removal
Retard normal strength gain
Endanger safety of the structure

Having assured ourselves that the concrete is of the required quality the following procedures are considered essential to obtain the desired results in performance under varying weather conditions.

May it be repeated that the conditions and procedures discussed here were predicated on New York weather. But in as much as they are based in general on temperatures and time, they should be generally applicable throughout the United States or any other location in this temperature-humidity range.

COLD WEATHER

It is the purpose of these recommended practices to:

1. Protect concrete from freezing during the early hardening stages;
2. Insure adequate strength gain of the placed concrete;
3. Maintain normal job progress during cold weather.

Temperature of the Concrete at Time of Placing.—Irrespective of source, at the time concrete is actually deposited in forms it shall have a temperature

not lower than 50 F and not higher than 80 F (the New York City Building Code makes it unlawful to despoit concrete with a temperature lower than 50 F).

Aggregates.—Coarse and fine aggregates shall be free of frost and ice particles. Aggregates must be heated when their temperature is below 25 F.

Mixing Water Temperatures.—Care shall be exercised to prevent direct contact between hot water and unmixed cement.

Water introduced at the site to a previously unmixed dry batch shall not exceed 150 F and shall be added only when the mixer is rotated.

There shall be no limit to the temperature of water when added only to aggregates with no cement in the mixer, provided the resultant temperature of the concrete does not exceed 90 F.

Admixtures.—Admixtures and/or cement substitutes (when use is permitted by local codes and approved by the engineer), should be used in quantities determined by actual job experiments.

Temperature Records.—Temperature readings should be recorded showing the date, hour, outside temperature, and temperature at several points within the enclosure, to show the most favorable and unfavorable conditions to which concrete is subjected.

Temperature readings should be taken at the start of the work and at least every four hours thereafter, so long as temporary heat is maintained.

After temporary heat has been discontinued the outside air temperature should continue to be measured and recorded. This procedure should be followed with all types of concrete.

Preparation of Forms.—All forms or surfaces to receive concrete should be heated above the freezing point and be completely free of frost, snow and ice.

Maintaining Temperatures After Placing Concrete.—When the lowest expected outside air temperature is above 35 F, and subject to the approval of the engineer, no temporary heat or cover is needed, provided the concrete temperature does not fall below 50 F.

When the lowest expected outside air temperatures are from 35 F to 28 F no temporary heat shall be required when the lowest expected concrete temperature does not fall below 50 F, but adequate cover must be provided.

This cover may be in the form of canvas, plastics, fabric, or kraft paper, together with a layer of salt hay 4 in. thick, or an approved type of insulated blanket giving special care to the protection of edges and corners.

Due consideration must be given to thin wall sections and wind exposures which may require additional protection.

Covers shall remain in place for 72 hr after concreting has been completed for regular cement - 36 hr for mixes using accelerators, high early strength cement or additional cement (at least 15%).

Where lowest expected outside air temperatures are from 18 F to 28 F, it shall be mandatory to provide both enclosures and temporary heat.

With regular portland cement the temporary heat shall be continuously furnished for 72 hr after completion of placing, in sufficient quantity to maintain a temperature in the concrete of not less than 50 F, or at least 36 hr when high early strength or additional cement or accelerators are used.

When lowest expected outside air temperatures are below 18 F no concrete shall be placed until such time as contractor's equipment for adequate heating and control has been approved.

When placing floors, in addition to the protection hereinbefore noted, it is recommended that tarpaulins, plastics, etc., used for housing and supported on

horses or other frame work, shall follow closely the placing of the concrete so that only a few feet of the finished slab is exposed to the outside atmosphere at any time.

Tarpaulins, etc., should be arranged so that the heated air from the story below can circulate freely in the space between the tarpaulin and the freshly placed concrete. Temporary openings may be left in the floor to provide for this.

Tarpaulins removed during working hours for continuation of the work, if the temperature is above 35 F, must be replaced on completion of working operations.

Methods of Temporary Heating.—Temporary heat may be provided by heated pipe coils or radiators, oil salamanders, coke salamanders, or blower type heaters.

No heating method should be permitted which will subject finished floors to excessive concentrations of heat, causing rapid drying or direct contact with combustion gases.

All exposed surfaces within the heated area should be wet down and maintained in a moist condition for curing and fire protection.

Adequate fire protection and watchman service should be maintained during temporary heating operations.

Insulated Forms.—Insulated forms may be used with the approval of the engineer on demonstrated degree of protection which will be provided thereby.

Curing.—Concrete must be protected from water loss. This can be accomplished by application as soon as possible without harm to the concrete surfaces, of exhaust steam, vapor resistant paper, polyethylene film or curing compounds.

Removal of Forms.—During cold weather removal of forms is very critical. Regardless of the minimum stripping times listed below, shores should not be removed before the concrete is strong enough to withstand its own weight and any contemplated superimposed load. Reshores must be installed when loads from upper floors must be supported.

When using regular portland cement, floor slabs shall not be stripped in less than 5 days from completion of placement. This may be reduced to 3 days when adequate demonstration is made to satisfy the engineer that safety of the structure is assured.

Wall, column and beam side forms may be removed 24 hr after completion of placement.

When mix is made with accelerators, high early strength cement or additional cement (not less than 15%), floor slabs shall not be stripped in less than 48 hr after completion of placement. This may be reduced to 40 hr on adequate proof of safety.

Wall, column and beam side forms shall not be removed in less than 18 hr after placing.

HOT WEATHER

To protect concrete from detrimental effects of hot weather, such as strength reduction, loss of workability, cold joints and cracking, and to maintain normal job progress during hot weather, the following recommendations should be observed:

Temperature of Concrete at Time of Placing.—Irrespective of source, at the time concrete is actually deposited in forms its temperature should not be lower than 50 F and preferably 80 F, but never above 90 F.

Equipment Required.—During the period June 1 to October 1 the contractor should have available for ready use, adequate facilities such as wind breaks, fog nozzles, etc. to accomplish desired results.

Handling of Aggregates, Mixing Water, and Cement.

Aggregates.—Aggregates should be stored in locations protected from excessive heat. Stockpiles should be protected from direct rays of the sun. Stockpiles should be sprinkled and kept moist during batching operations. Wind breaks should be provided to avoid excessive drying.

Mixing Water.—Cool mixing water is essential. Pipe lines and storage tanks should be buried, insulated or painted white.

Crushed ice may be used to reduce water temperature but on completion of mixing concrete, ice should be completely dissolved.

Cement.—If temperature of cement when batched exceeds 170° F. it must be ascertained that the cement is not flash setting and direct contact between water and hot cement is prevented.

Admixtures and/or Cement Substitutes.—Admixtures and/or cement substitutes shall be used where consistent with local codes and approved by the engineer. Quantities should be determined by actual experimentation with specific materials involved. Admixtures and/or cement substitutes that reduce water and retard set may be helpful to delay early setting and maintain strength. Admixtures and/or cement substitutes that have an accelerating effect should be used only under careful supervision.

Preparation of Forms.—All forms or surfaces, including sub-grades, reinforcing steel, etc. to receive concrete should be protected against excessive air currents and should be sprinkled systematically with cool water. Wetting down area around the work will cool surrounding air and increasr the humidity, thus reducing temperatures and evaporation from the concrete.

Production and Delivery.—In addition to handling aggregates, water and cement as previously noted, the loss of slump should be kept at a minimum by minimizing the lapse of time between mixing and placing, avoiding delays in batch mixing and truck dispatching, and organizing job conditions and equipment to prevent additional mixing, and the use of approved water reducing retarders.

Concrete which has lost one third of its original slump should be placed only when procedures have been adopted which will insure proper placing. Retempering by addition of water should not be permitted.

Placing.—Adequate skilled crews should be available to handle and place the concrete immediately upon delivery. Dry contact surfaces should be wet down before commencing placing of concrete.

Temperatures of surfaces to receive concrete should approximate the temperature of the concrete being placed.

Evaporation of water from freshly placed concrete should be held to a minimum by shading of operations, reducing air circulation in area of operations and maintaining fog spray during ioerations.

Cold joints should be avoided by providing trained personnel to handle and place the concrete immediately after delivery to the forms at an acceptable temperature, and by placing in layers thin enough and areas small enough so that vibration or working of concrete will insure complete union of the adjacent layers.

Lengthening time of setting by use of approved water reducing retarders or placing a bulkhead at suitable point where placement is stopped temporarily.

Protection and Curing.—Continuous water curing should be maintained, starting immediately after finishing, for a minimum period of 24 hr on formed or unformed concrete.

After 24 hr curing may be continued by continuous water curing, application of an approved plastic membrane, spraying with an approved curing compound (preferably white pigmented).

On completion of moist curing, every effort should be made to reduce the rate of drying by avoiding air circulation.

CONCLUSIONS

It has been the purpose of this paper to provide a simple set of suggested rules for concreting during hot or cold weather, and ways for accomplishing them.

It is felt that these rules, when properly followed, will eliminate most major hazards of concreting during out-of-the-ordinary periods of high or low temperatures in generally temperate zones.

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CONSTRUCTION OF ROCKY REACH HYDROELECTRIC PROJECT

By W. N. Evans,[1] F. ASCE and J. H. Boyd,[2] M. ASCE

SYNOPSIS

Rocky Reach Dam was built in four steps in areas protected by berms and circular sheetpile cells, as much as 75 ft high, installed to rock through boulders. The million cubic yards of concrete was made from locally available gravel by four 2-cu yd tilting mixers; it was transported in 4-cu yd buckets on semi-trailers to big whirler cranes on 100-ft high gantries for placing in heavily-built panel forms. Rock bolts were used to tie down a section of the dam in which fault planes were encountered. Final river diversion was effected by connecting two big rocks by cable and dropping one rock upstream as an anchor and the other into 35-fps river flow.

INTRODUCTION

The Rocky Reach Hydroelectric Power Project is located on the Columbia River about 7 miles above the City of Wenatchee in North Central Washington. There, the Columbia River flows in a tremendous canyon nearly a mile wide and 2,000 ft deep. The site chosen for the dam is one at which the river is a relatively shallow, fast flowing stream near the base of the west wall of the canyon and at which exploration had determined that bedrock in the stream reached its highest level. The general arrangement, as shown in Fig. 1, was chosen in order to provide space for a powerhouse that would accommodate the number of units proposed and for a spillway that would pass maximum floods.

Note.—Discussion open until April 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Construction Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. CO 3, November, 1960.

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Beginning at the west bank, a 450-ft long non-overflow forebay wall of concrete gravity section closes the gap between the powerhouse and the rock bluff. The 1,100 ft long powerhouse includes a service bay at the south end and space for the installation of eleven hydraulic turbine driven generators, seven of which are being installed. These seven will have a total peak generating capacity of 775,000 kw when operating under a 93-ft gross head. Total peak generating capacity, when all units have been installed, will be 1,215,000

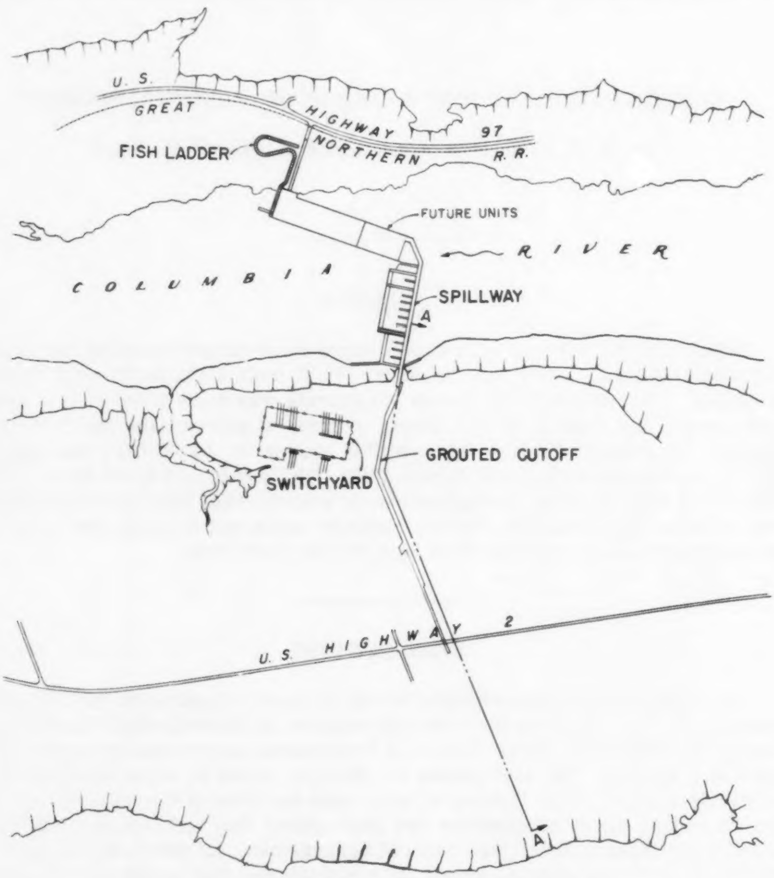

FIG. 1.—ROCKY REACH HYDROELECTRIC POWER PROJECT

kw. Adjoining the powerhouse at its north end, and connecting it to the spillway section, is an L-shaped non-overflow concrete gravity-arch section 375 ft in length, called the center dam. The spillway section, that extends 750 ft across the river, provides 12 bays, each having a 50 x 58 ft clear opening, and is capable of passing river flows of up to 1,100,000 cfs. This is approximately $1\frac{1}{2}$ times the 718,000 cfs maximum flood of record that was experienced at

the site in 1894. The east abutment, 120 ft in length, is again a concrete gravity section and completes the concrete portion of the dam.

East of the concrete structure, the floor of the canyon is a broad terrace that extends more than 3,500 ft from the abutment to the canyon wall. Elevation of this terrace is generally about 750 ft above sea level or about 40 ft above maximum pool elevation of 710 ft. As shown in Fig. 2, the terrace consists of a deposit of pervious gravels, of varying thickness up to 125 ft, overlying bedrock. Above this gravel deposit are varved clays again varying in thickness from a minimum of 70 ft to a maximum of 125 ft. The contact surface between the varved clays and the pervious gravels below, slopes downward in an easterly direction and reaches the bedrock about 1,800 ft east of the east abutment of the dam. The top of the varved clays is at approximate el 665 ft or about 45 ft below maximum pool level. Overlying the varved clays to a depth of about 75 ft is another pervious gravel layer.

To provide a cutoff through the terrace, the lower gravels were grouted, using cement and chemical grouts, along a line from the east abutment concrete to the point at which the lower surface of the impervious varved clays intersects bedrock. A 2,000 ft long clay core earthfill dam was constructed on top of the varved clays to above maximum pool level to control seepage in the upper gravels.[3]

Federal Power Commission License No. 2145 for construction of the Rocky Reach Project was issued on July 12, 1956, and accepted by the owners, Public Utility District No. 1 of Chelan County. The license required that construction be started within 1 yr and be completed within 7 yr or by 1964. Since financing of the work was to be accomplished by sale of the District's own revenue bonds, the start of construction work was predicated on the prior completion of financing. To get the work under way as quickly as possible, it was decided to obtain financing and construct the project in two stages.

A contract for performance of stage I work was awarded on September 19, 1956. Stage I work included concrete construction for 9 of the 12 spillway bays and the east abutment, grouting work and construction of the impervious cutoff in the east bank terrace, and protection of the river bank upstream and downstream of the dam. Most of the work under the contract, except for the work on the east bank terrace, was completed by December, 1957.

Second, or final, stage work included construction of the three remaining spillway bays, center dam, powerhouse and forebay wall, and installation of all mechanical and electrical work. Included also was remaining concrete work required to raise the ogee sections of the Stage I spillways, that had been left 40 ft below final elevation for river diversion during performance of final stage work. Two final stage contracts, one a general contract and the other for electrical work, were signed in November, 1957. Although work being done under these two contracts is still in progress, it is now anticipated that the first unit will go on the line in July, 1961 with other units following in succession until the seventh is completed in February, 1962.

RIVER DIVERSION

Perhaps the major problem that confronts engineers and contractors involved in the construction of a dam on the Columbia River is diversion of the river. The cofferdam design must satisfactorily control the river to permit the maintenance of the schedule set up for performance of the work.

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3 "Design and Construction of Rocky Reach Grouted Cutoff," by W. F. Swiger.

The river itself dictates the timing of the work. Its flood cycle is quite regular. From September through mid-April flow at Rocky Reach is controlled by dams upstream that regulate the flow to 100,000 cfs or below, if possible, with a minimum of about 60,000 cfs being usual. Beginning in late April, the river starts to rise and reaches its peak flow usually in late May or early June. Peak flow exceeds 500,000 cfs at Rocky Reach about once in 6 years on the average. The maximum flood of record is 718,000 cfs, and the maximum flow experienced during the construction period to date was 423,000 cfs, in 1959.

Cofferdam work affecting the main flow of the Columbia River must be accomplished within the low water period of about seven months and must be undertaken with the full realization that, if not completed in that time, the entire project may be delayed a full year. This was the basis on which the schedule of work for the Rocky Reach Project was established.

In the bidding documents for both stages of work, the engineers had included a proposed method of diverting the river. Final plans adopted for construction of all diversion structures and the design on which the plans were based were, however, made the responsibility of the contractor.

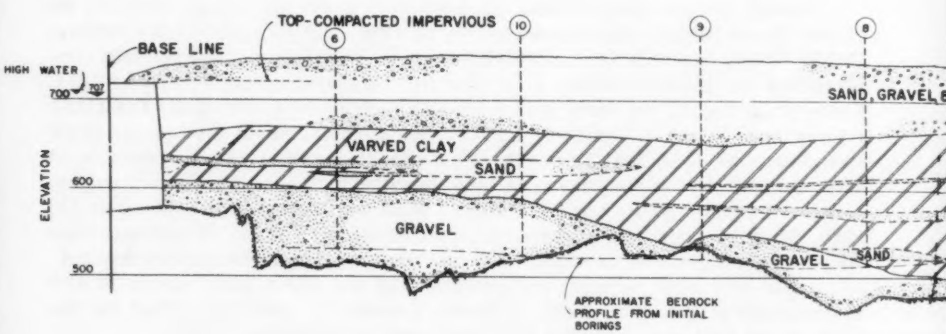


FIG. 2.—SECTION THROUGH

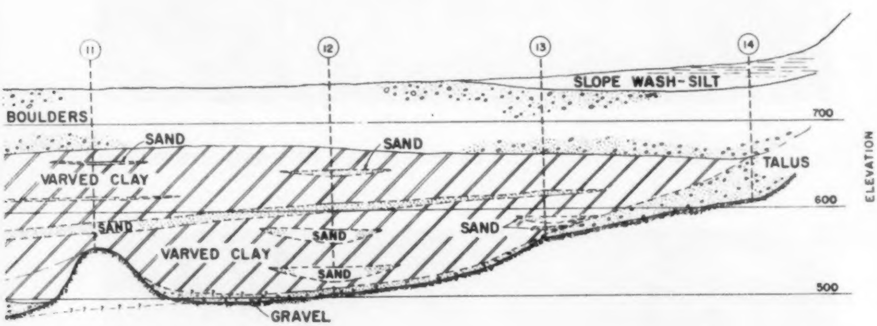
For Stage I, the contractor used a cofferdam that diverted river flow toward the right bank. Circular steel cells were used in the cofferdam to leave maximum river cross section for passage of flood waters. Rock groins on the outside and gravel groins on the inside were used to give additional stability to the cofferdam that, at some locations, was 75 ft high. Twenty-four $60\frac{1}{2}$-ft diam cells, spaced 64 ft on centers, were required to complete the cofferdam. Cells were made up of MP 101 or equivalent steel sheet piling and were joined together with small radius connectors also made up of steel sheets. End cells on the upstream and downstream legs were set into the river bank.

Fig. 3 shows the cofferdam for Stage I construction nearing completion. The rock groin along the upstream arm of the cofferdam provided slack water during construction. The cells were erected in the slack water.

Whereas overburden in the stream was shallow, it contained numerous boulders that would interfere with driving of piling. Therefore excavation to rock was considered necessary at each cell to assure proper seating of the

sheets. The height and diameter of the cells and the lack of the usual support from overburden created problems in stability during erection. These were overcome through use of the self-supporting internal ring-type template. Fig. 4 shows the self supporting internal ring type template used for racking sheet piling during construction of $60\frac{1}{2}$ ft diam cells. The template consisted of two structural rings, spaced 18 ft apart vertically, and supported by five 16-in. diam pipe spuds. After excavating for each cell, the template was set and leveled. The sheets for a complete cell were racked around it and all were lightly driven to seat them. A bottom seal of silty clay was then placed immediately over the rock by clam shell, and the balance of the cell was filled with sand and gravel.

Because of the critical stability of cells without fill, it was necessary to exercise extreme care while placing fill in any cell to assure that the cell was always uniformly loaded. This was done by placing the fill always at the center of the cell using a clamshell bucket or a chute. The cells became stable when about 2/3 filled and the template was then removed. The same care was exercised in completing the fill.



EAST BANK TERRACE

Work was started simultaneously on the upstream and downstream arms of the cofferdam, and each cell was completed before erection of the next cell was started. Completed cells provided a work base for equipment during the construction of the succeeding cell.

The stage I cofferdam was designed for 500,000 cfs flows and successfully withstood the peak flow of 415,000 cfs experienced during the 1957 high water season with only normal leakage.

Because of some delay in financing, the contract for performance of final stage work was not signed until November, 1957. With 2 months of the low water season already past, it became imperative to utilize every moment of available time to make a river diversion that could be ready for the 1958 high water season. The joint venture of contractors, who were then completing the Stage I work, together with one additional participant, were the low bidders and were awarded the general contract for the Final Stage. They already had

FIG. 3.—COFFERDAM FOR STAGE 1 CONSTRUCTION NEARING COMPLETION

FIG. 4.—SELF SUPPORTING INTERNAL RING TYPE TEMPLATE

men and equipment at the site, and an existing Stage I cofferdam that provided a base for further work in the river.

The successive steps of cofferdam construction for Stage II, or the Final Stage, are shown in Fig. 5.

Twenty-one circular cells were used in the river where the cofferdam would be subject to high velocity flow. These were of the same diameter and were constructed in the same manner as for Stage I. They were connected to the shore by two dikes.

Seven cells were erected first in the main channel of the river, working in the shelter of a rock groin extending out from the Stage I cofferdam. Dumped rock dikes for shore connections of the main cofferdam were started on the west bank, and removal of the Stage I cofferdam was then begun. To permit starting with the powerhouse excavation before completing the main

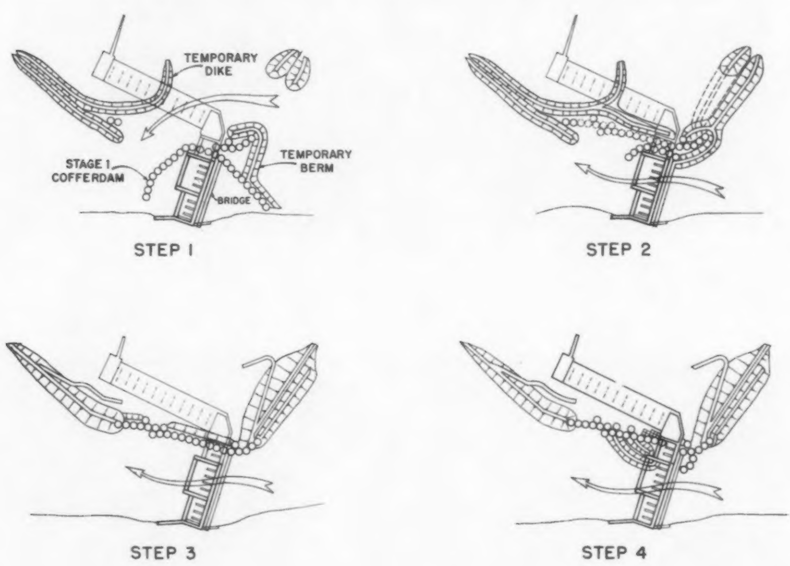


FIG. 5.—COFFERDAM CONSTRUCTION

cofferdam, the contractor subdivided the work area with a temporary rock fill inner cofferdam with an impervious core. The inner cofferdam allowed a portion of the powerhouse area to be unwatered early. Fig. 6 shows the cofferdam for final stage work under construction. Primary closure was made at the gap in the right background. The temporary dike near the center of the photograph permitted the contractor to begin excavation before the main cofferdam was completed. Fig. 6 also shows construction of the upstream dike of the main cofferdam. This dike was more than 900 ft long and in some sections 75 ft high. It was constructed by dumping a rock groin out from the west bank to connect with the seven cells that had been constructed from the Stage I cofferdam. This groin made the primary closure of the cofferdam and diverted the Columbia River through the Stage I spillway sections for the first time. Velocities of up to 35 fps were encountered at the closure. To overcome

FIG. 6.—COFFERDAM FOR FINAL STAGE WORK UNDER CONSTRUCTION

FIG. 7.—COFFERDAM FOR STAGE 1 CONSTRUCTION AFTER UNWATERING

such velocities, the contractor used pairs of large pieces of quarried rock drilled and tied together by cable. These rocks weighed up to 25 tons. One rock was dumped upstream of the groin to act as an anchor, and the second was dumped into the gap.

To complete the upstream and downstream dikes, second rock groins were constructed parallel to the original groins, and the area between was cleaned to bedrock with draglines. Sand was then dumped in the area between the groins, and steel Z-piling was driven through the sand to bedrock to provide a positive water cutoff. The piling cutoff was used instead of impervious fill as it was considered more positive and required less width of dike.

Where space was available, pervious rock berms were placed as backup for cells, both inside and outside, as had been provided in Stage I. Where space was not available, backup cells similar to cells in the main cofferdam were provided. Near the center of the cofferdam the cells were so arranged as to provide for a fish trap. Cement grout was used to cut down leakage under cells and through known rock "guts" beneath the dikes.

Two main pumping stations, each provided with five 20-in. 15,000 gpm turbine pumps, were installed within the cofferdam, and all leakage into the dam was directed to these. Following initial dewatering, only a minor part of this installation was ever required to maintain dry working conditions.

The cofferdam was completed in time to take care of the 1958 high water season when flows reached a maximum of 390,000 cfs. Observations during the successful passage of this flow, and a subsequent maximum flow of 423,000 cfs in 1959, indicated that flows of 500,000 cfs for which the cofferdam had been designed could have been passed safely. A more detailed description of the river diversion is contained elsewhere.[4]

Construction of the final three spillway bays that adjoin the Stage I work was scheduled for the 1958-59 low water period. Circular cells were extended from the main cofferdam to the end of spillway block 4. Placing of the concrete for these bays was completed, and the temporary cells were removed so that water flowed through the bays during the 1959 flood season.

Following the 1960 flood season, the final stage cofferdam will be breached and the river will be permitted to flow through the intakes and draft tube spaces for the four future units. This will permit completion of concrete work on the spillway ogee sections that purposely had been left low for river diversion. In the spring of 1961, when all spillway concrete has been placed and the tainter gates have been installed, the future unit intakes will be closed off with concrete stop logs, and water will be permitted to flow over the spillway.

During design of the Project, an arrangement was made with the R. L. Albrook Hydraulic Laboratory at Washington State University at Pullman, Washington, to perform model studies of various hydraulic features. A comprehensive model of the project at 1:75 scale was built for use in these studies. The model was also used for investigation of all diversion schemes before they were constructed. Excellent agreement was obtained between model and prototype velocities and water elevations.

DEWATERING EAST BANK

The major problem encountered in the construction of Stage I was the control of seepage from the east bank into the cofferdam. The lower gravel

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[4] "Pushing the Columbia River Around," by L. E. Dixon, Western Const., November, 1959, pp. 45-49.

strata proved to be considerably more pervious than had been anticipated. The depth and nature of the gravel strata and the overlying varved clays between the arms of the cofferdam made it impractical to use a sheet pile cutoff wall to control the water. It was decided, therefore, to install a line of 19 wells on the east bank, back from the area to be excavated, between the upstream and downstream arms of the cofferdam with the idea of lowering ground water levels and thereby reducing seepage into the work area. To assist in controlling ground water, and as a part of the permanent protection of the left bank, a sheet steel cutoff wall was constructed from the upstream arm of the cofferdam, a distance of 700 ft upstream and parallel to the river bank. A temporary sheet pile wall was constructed downstream from the downstream arm, a distance of 300 ft. The 19 wells had a total pumping capacity of about 30,000 gpm. It was found necessary, however, to construct, in addition, a deep sump within the cofferdam. Eighteen pumps with total pumping capacity of 160,000 gpm were installed in this sump. Fig. 7 shows this cofferdam dewatered. The main sump is also shown as is the header for the seepage wells. The line of wells at the top of the excavation and the bank of pumps in the sump maintained dry working conditions.

STAGE II EXCAVATION

A total of 750,000 cu yd of material were excavated for the powerhouse and center dam structure. The bulk of this material was located within the banks of the original river and could not be reached until cofferdams had been constructed. Thus, only minor quantities from the structure excavation could be used in cofferdam construction, and the bulk was deposited downstream of the site at which it was used for construction facilities and will form a permanent useful level area.

Conventional equipment consisting of crawler type self-propelled wagon drills and jack hammers were used for drilling. Twenty-two ton end dump units that were loaded by $2\frac{1}{2}$ and $3\frac{1}{2}$ yd shovels removed excavated material to the disposal area.

Rock at the site is a biotite gneiss and includes numerous fractures along cleavage planes. The majority of these fractures are tight, but some show past slippage with gouge zones varying from a few inches to several feet wide. These shear planes posed several problems.

Originally, the center dam had been conceived and designed as a gravity section dam. However, when rock excavation had progressed to the area of the center dam it was noted that several tight fault planes in the rock dipped at an angle of approximately 54° below horizontal toward the powerhouse or generally in the direction of greatest horizontal hydrostatic thrust from the reservoir. Immediately below the toe of the center dam it was necessary to excavate for draft tube No. 11, one of the future units.

Additional rock was removed to form a stepped terrace beneath the center dam and more than 100 rock bolts, 30 ft long, were grouted into holes drilled into the rock at right angles to the bedding planes to provide additional stability at this location. The center dam was also re-designed to act as a modified gravity arch section to reduce the shear load transferred from the center dam into the rock beneath it. Thrusts developed by the arch are delivered to gravity abutments formed by the spillway sections on one end and the base of the unit intakes at the other. To assure arch action, grout stops and shear keys were provided in all contraction joints between Unit 10 of the powerhouse and

block S3 of the spillway, and the arch contributing lower portion of these joints will be cement grouted under pressure prior to raising the lake.

In excavating the draft tubes of future units 8 and 9, an area was found in which two nearly parallel faults intersected, resulting in a zone of severly sheared and broken rock (see Fig. 8).

This fault zone dipped about 45° to the west with a strike making an angle of about 59° with the axis of the powerhouse. Gouge and sheared rock was as much as 12 ft thick in places. This unstable material extended under the sound rock on which the heavily loaded main powerhouse piers and intermediate draft tube piers were to be built. The fault zone was followed down from its surface exposure in the excavation to depths within reach of conventional excavation equipment. To reach greater depths a series of 3 calyx holes, 36 in. in diameter, were drilled through the rock into the zone. The gouge was then mined out under the piers and replaced with concrete. Grout pipes were installed and all shrinkage cracks were filled with grout under pressure.

CONCRETE

Gravel deposits suitable for concrete aggregate are abundant through the Columbia River Valley. After testing materials from various potential borrow areas, it was decided that material from the east bank terrace immediately adjacent to the left abutment of the dam could be processed to provide aggregate of the proper quality. The test program had revealed that material at the site was generally sound and, with the addition of a blend sand available from a pit about one mile south of the site, the required gradation could be obtained. A substantial proportion of the aggregate from a thin zone at a depth of about 10 ft had surface coatings of calcium carbonate with some opal and, as such, were potentially alkali reactive. To remove the coatings and any soft materials that might be included, all aggregate was processed through a rotating drum scrubber. Also, Type II Low Alkali cement was specified for all concrete.

The Stage I contractor elected to erect his aggregate processing plant and his concrete batching plant immediately adjacent to the aggregate source. Fig. 9 shows the layout of the aggregate processing plant and batching plant. Conveyors moved processed aggregate from a reclaim tunnel beneath stockpiles to the batching plant.

At the processing plant pit run, aggregate was run through a jaw crusher and sized by primary and secondary screening units to provide graded coarse aggregate ranging from No. 4 minimum to 6 in. maximum size. The graded coarse aggregate was then stockpiled for later selection and use in four ranges of gradation, No. 4 to 3/4 in., 3/4 in. to $1\frac{1}{2}$ in., $1\frac{1}{2}$ in. to 3 in., and 3 in. to 6 in. Fine aggregate was produced by further crushing and washing of materials selected from the secondary screening units and adding the blend sand hauled from off site. The scrubber, that also served as a classifier, was located between the primary crusher and the primary screening unit. It consisted of a 5-ft by 22-ft revolving cylinder with an interior concentric plate screen sized to reject plus 6 in. material for further crushing. Angle bars mounted on the interior perimeter of the cylinder, parallel to its longitudinal axis, tumbled the aggregate to abrade and loosen surface coatings that were then washed from the unit using water jets. Periodic tests on processed aggregate indicated that the scrubber unit has satisfactorily removed the surface coatings to provide a sound non-reactive aggregate.

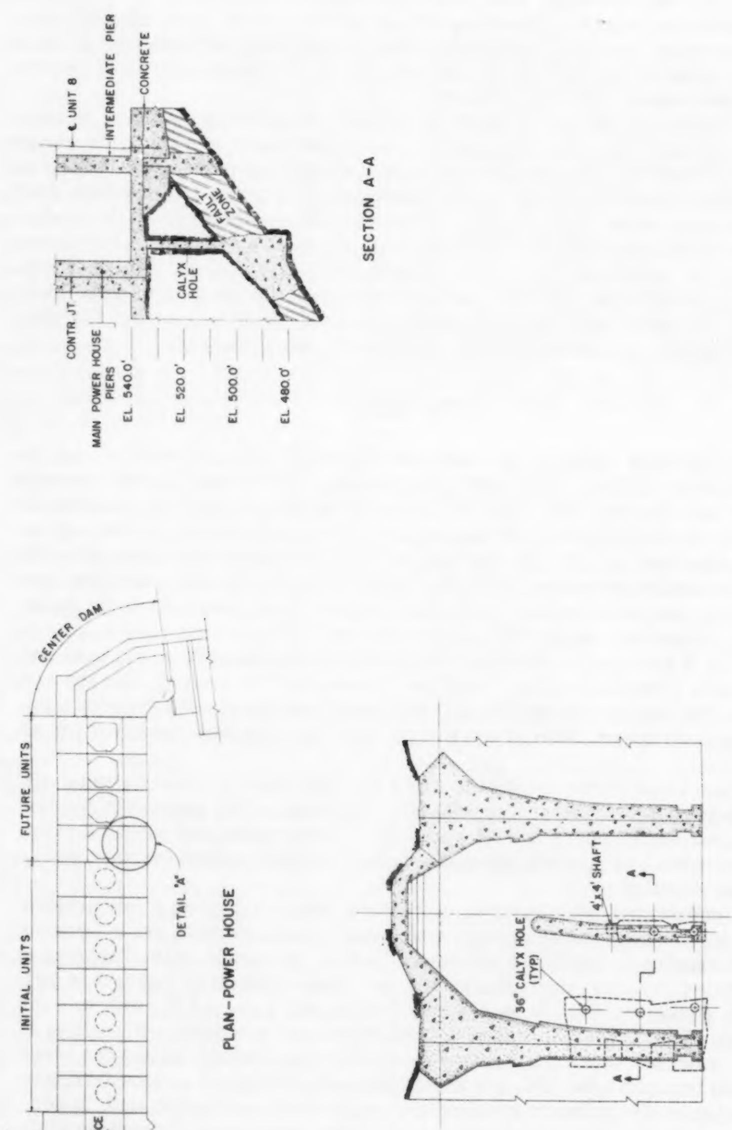

FIG. 8.—FAULT ZONE AT FUTURE UNIT NO. 8

FIG. 9.—LAYOUT OF AGGREGATE PROCESSING PLANT AND BATCHING PLANT

FIG. 10.—PLACING CONCRETE DURING FINAL STAGE CONSTRUCTION

Processed aggregate was selected as required from a reclaim tunnel beneath the stockpiles and conveyed by belt through a rewash unit to the concrete batching plant. The batching plant used during Stage I construction consisted of three 2-yd tilting mixers controlled for automatic and simultaneous batching of any of 12 predetermined mixes. The same plant, with the addition of a fourth 2-yd mixer, was used for final stage work. Capacity of the plant with the fourth mixer in operation was about 200 cu yd per hr. This capacity provided ample reserve as maximum yardage placed in any one day was 3,410 cu yd. Silo storage for about 7,000 barrels of cement was provided at the batch plant. All cement was transported in bulk to the site by truck.

During Stage I construction, concrete was transported from the batch plant to the placing site by a fleet of highway tractors pulling semi-trailers each carrying two 4-yd buckets. The trailers provided space for three buckets so that when any truck arrived at the placing site the handling crane could place a recently emptied bucket on the vacant space on the truck and grab a full one. This allowed for continuous operation of the unloading crane when trucks were moving into and out of unloading position and reduced the round trip time of each truck. Buckets were lifted by crawler cranes with 140-ft booms. To further reduce unloading time and complicated hose or rope connections, the contractor devised automatic grab and release hooks operated by either air or hydraulic cylinders mounted above the hook. Rotation of sheaves above the hook charged the cylinders in normal operation and opening or closing of the hook was controlled by pressure on a lever mounted on the upper inside of the hook. Every other operation of the lever opened the hook and every other operation of the lever closed the hook. With little practice, crane operators could open and close the hooks without hesitation or lost motion. Fig. 10 shows a typical concrete placing operation during performance of final stage work.

Although the final stage work was located mainly on the right bank and contained approximately 800,000 cu yd of the 1,025,000 cu yd of concrete required for the entire project, the Contractor, when awarded the final stage work, decided to maintain the batching plant he had used during construction of Stage I and again transport concrete by truck, this time across the river via a temporary bridge. The cost of hauling concrete this greater distance was easily offset by the savings made in eliminating the need of relocating the batch plant to the right bank and the need of transporting aggregate to such a plant. The bridge also provided communications across the river and made available the more spacious working and storage areas on the left bank. The bridge, consisting of a wood deck supported on 70-ft steel plate girders on concrete piers, was constructed in the dry behind the Stage I cofferdam and was located immediately upstream of the dam as shown in Fig. 11. It provided good service until the night of June 4, 1958, when, with the flow in the Columbia near its 1958 peak of 390,000 cfs, three piers of the bridge near its east end were lost (see Fig. 12.).

Each pier in the spillway section of the dam is stepped upstream at el 640 to provide seats for a permanent gate maintenance caisson and space for guides for the temporary stop logs used during placement of concrete in spillway ogees. To repair the bridge, the contractor supported structural steel brackets off these pier noses, with tension ties back to the spillway piers, as shown in Fig. 13. The permanent spillway bridge beams that had already been delivered to the site were erected on these brackets and the bridge was returned to service in 28 days. While the bridge was out of service, concrete operations were carried out on a limited basis by hauling the

FIG. 11.—CONSTRUCTION BRIDGE

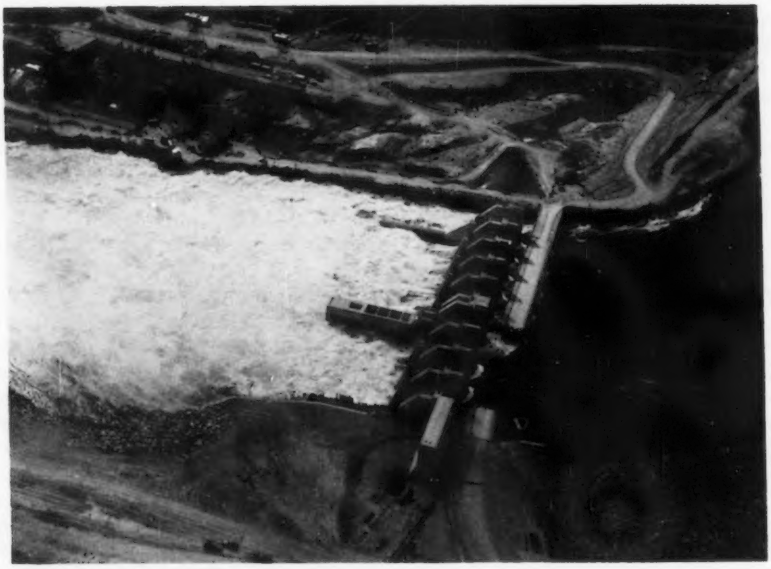

FIG. 12.—LOSS OF THREE PIERS OF THE CONSTRUCTION BRIDGE

concrete from the batch plant to the placing site across the river through Wenatchee, a total of 21 miles.

The flexibility of truck haul over rail or conveyor haul of concrete was well demonstrated in this instance. Concrete delivered to its site after this long haul suffered only slight losses in slump and entrained air and no noticeable segregation or loss in compressive strength.

For placement of final stage concrete, the contractor provided four whirler cranes, rail mounted, with structural steel gantries. Fig. 14 shows the whirlers along the upstream and downstream sides of the powerhouse provided crane coverage of the powerhouse and center dam for placing of concrete. Study indicated that gantry type whirlers with a maximum gantry height of 100 ft could reach all parts of the structure without requiring a trestle for the track system. On the intake side of the powerhouse, the track was laid as close to the intake structure as was practical with the low side of the track being on a fill supported by a 12 in. x 12 in. timber crib to prevent the fill from sloping into the concrete area. The opposite rail, in most cases, was built on solid ground. The intake track paralled the powerhouse and followed the upstream face of the center dam, first on a curve with a 220-ft centerline radius and then tangent to the spillway, and ended at approximately the west end of the spillway. The whirler type gantry cranes used on the intake side of the powerhouse were equipped with 100-ft gantries and 157-ft booms and had a capacity of 11 tons at 145-ft radius.

Whirler tracks on the draft tube, or downstream, side of the powerhouse, were built substantially the same as those on the intake side with the exception that concrete retaining walls were used in lieu of cribs. The retaining walls were built to the alignment of the centerline of the rails, so that the rails were placed directly on top of the concrete, and no ties were needed. This track accommodated two whirler type gantry cranes with 80-ft gantry heights and 140-ft booms. Both machines were of the same capacity as those used on the intake side.

The four whirlers were augmented by crawler cranes of similar capacity as the whirlers. These proved especially useful in the lower levels of the powerhouse. They also placed all of the concrete in the forebay wall and fish ladder structure, as both were out of reach of the powerhouse gantry cranes. As concrete reached greater heights in the powerhouse, use of the crawler type machines was discontinued because they couldn't reach desired heights.

When no longer required at the powerhouse, the two whirlers from the upstream side were dismantled and re-erected on temporary steel girders spanning the spillway bay openings. There they will be used in placing concrete for the spillway ogees and in installing the tainter gates.

All forms were designed for the use of heavy hardware (ties) spaced at the maximum possible distance apart. This, of course, produced quite heavy form sections, and required the use of power equipment for stripping and raising. Steel framing was used in forms in some cases, but all contact surfaces were sheathed with either tongue and groove lumber or plywood. Concrete lifts in the powerhouse varied from 5 ft to 15 ft, so standard panels were built varying in height within these limits.

Generally, forms were designed to match concrete placing rates. That is, if a 14-ft lift of concrete in a wall, for example, was congested with steel, that would affect the rate of concrete placing, forms were designed to accommodate this situation rather than to carry the full plastic load. With the exception of those for the draft tubes, intake and scroll case soffits, forebay

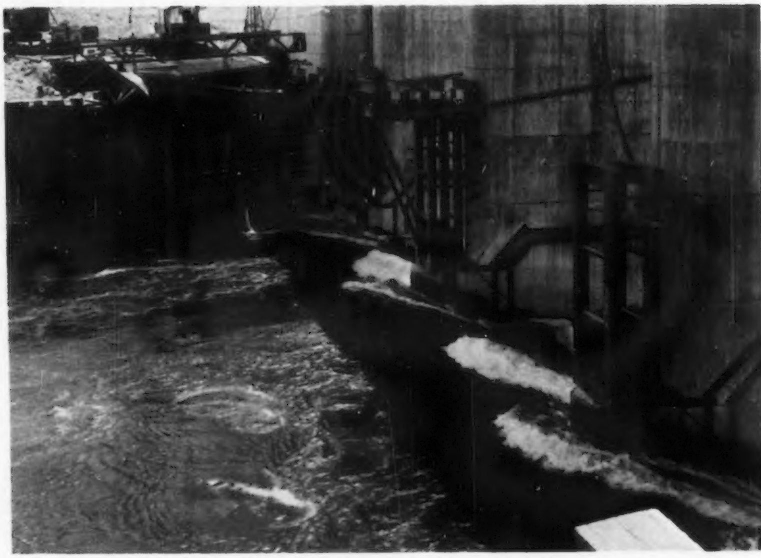

FIG. 13.—STRUCTURAL STEEL BRACKETS AND THE TIE BACKS AROUND THE SPILLWAY PIERS

FIG. 14.—WHIRLERS ALONG THE UPSTREAM AND DOWNSTREAM SIDES OF THE POWERHOUSE

and center dams, all forms were made of wood. Wherever possible, standard panels were designed in order to provide as many re-uses as possible.

At the time of purchase, the Rocky Reach turbines were second only to the units at Grand Coulee in horsepower rating and were among the largest on this continent in physical size. Fig. 15 shows an interior view of the concrete scroll case for one of the units. The section of concrete scroll case shown here is over 30 ft high. The men in the foreground give an impression of the size of the units.

As the powerhouse units were similar, and the construction schedule allowed adequate time, only a single set of specially designed draft tube, scroll case and intake soffit forms were used. All of these were constructed of steel and sheathed with plywood. Sections were made the maximum size possible complete with attached steel shoring. They were cycled from one unit to another on a systematic schedule. Fig. 16 shows a section of the shoring and forming for the scroll case roof of one of the units. Shoring and forms were designed for re-use in successive units. Such shoring and forming proved to be economical as well as expeditious. Its use eliminated the need of reforming of intricately warped surfaces and the usual forest of temporary shoring, customarily provided for each of the powerhouse units. As other formed surfaces, such as walls of the typical units, required considerably more time, it was generally found that two complete sets of forms for these areas were required to match the time cycle of the single set of draft tube, scroll case and intake soffit forms mentioned previously.

Gravity sections of the structure, such as the center dam and forebay wall were formed with standard type 5-ft steel cantilever panels sheathed with wood. The special forms for the intake soffits, draft tubes and scroll cases were fabricated off site. Wood forms were prefabricated in the job form shop. Thus, practically all built-in-place form work was eliminated from the job.

No major problems arose during the placing of the final stage concrete. The contractor's plant and equipment were adequate to produce and place concrete as quickly as space limitations would permit form work to be prepared to receive it. During the 9 month period from February 1, 1959 to November 1, 1959, a total of 400,000 cu yd were placed. Deducting from this period a three-week shut down because of labor trouble, concrete was placed at an average rate of over 11,000 cu yd per week.

FISH HANDLING FACILITIES

As is required at most hydroelectric projects constructed on west coast streams, it was necessary to provide facilities for passage of fish moving upstream to spawn. Permanent facilities were made an integral part of the concrete structures. Entrances to a fish ladder for upstream migrants are provided at the spillway and along the entire length of the powerhouse. From the entrances, fish will pass to the forebay by means of channels and a fish ladder adjacent to the west bank. Location of the fish ladder is shown in Fig. 1. A fish counting station is located at the exit to the forebay and will provide photographic record of number and species. Attraction water for the fishway passages is provided by three hydraulic turbine driven pumps pumping a total of 3,525 cfs from the tailrace. Downstream migrants will be collected at the intake to the turbine pumps and conducted to the tailrace through large diameter conduits and over free falling wiers.

FIG. 15.—HYDRAULIC TURBINES

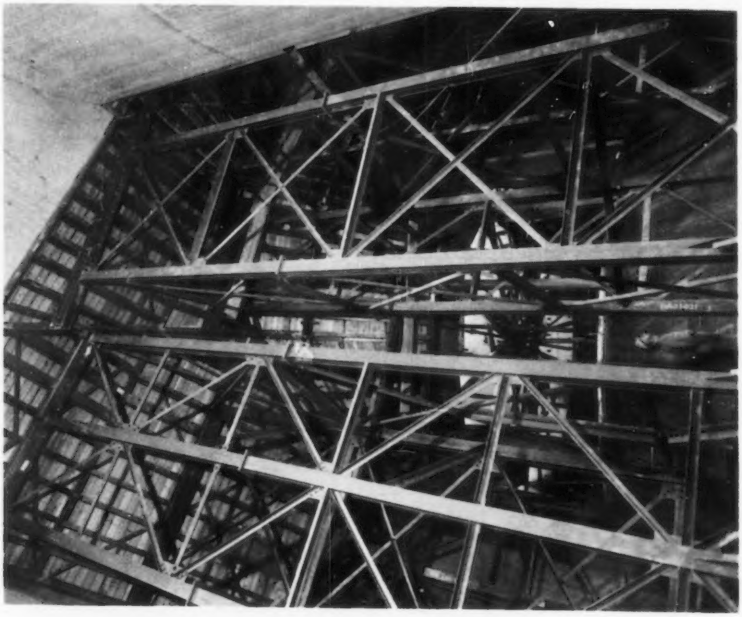

FIG. 16.—STRUCTURAL STEEL SHORING AND WOOD SHEATHED STEEL RIB FORMS

Temporary fish handling facilities had to be provided during the full construction period. These were required to accommodate each year the estimated 90,000 fish that would be passing upstream through the site. Also, the construction schedule itself had to be arranged so that once the fish began to move upstream their passage through the site would be uninterrupted. Since the river at no time was completely blocked off, the passage of downstream migrants was no problem. Various schemes were used, however, to accommodate the upstream migrants. During the 1957 spawning season, with the Stage I cofferdam in place, upstream migrating fish had to traverse the water passage between the cofferdam and the west bank. To reduce velocities in this section, that would otherwise be prohibitive, quarry rock was dumped along the outside of the cofferdam to form an uneven surface on the stabilizing

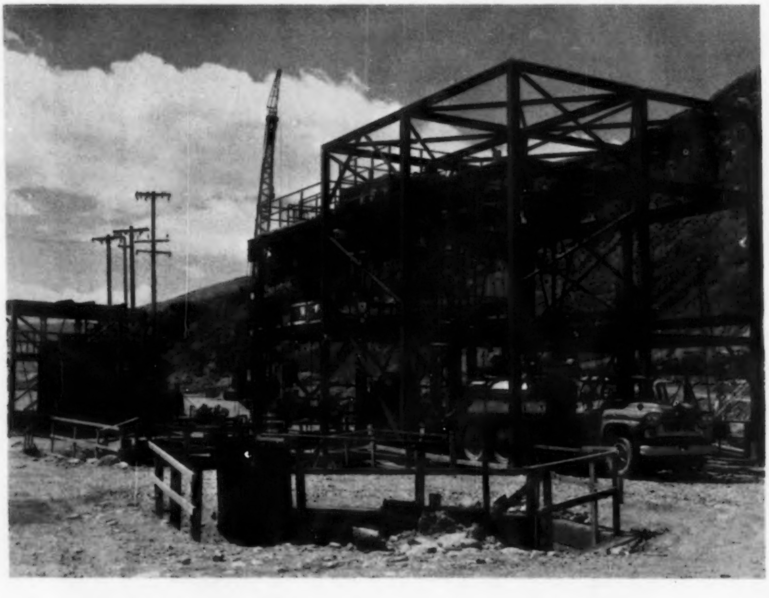

FIG. 17.—TRUCK MOUNTED FISH TRANSPORT TANK

rock groin. Velocities immediately adjacent to the groin were significantly reduced thereby, and fish were able to pass upstream.

Model tests showed that in the 1958 through 1960 seasons, while the final stage cofferdam was in place, velocities of water flowing through the uncompleted spillways would be an insurmountable obstacle to upstream migrants. A temporary fish ladder was constructed in the spillway bay nearest the east bank for fish moving upstream on that side of the river. The ladder was of the vertical baffle type and of a height sufficient to accommodate all stages of river flow.

For the west bank, a trap was constructed in the final stage cofferdam, the cells being so arranged as to support the necessary equipment and tanks. Holding ponds, and brailing and elevating equipment were arranged in a 45-ft diam steel tank in the cofferdam, and a fish passage was provided from the river through the cofferdam to the tank. A sheet piling deflector was erected extending into the river from the cofferdam cell immediately upstream of the fish passageway to provide a velocity barrier that would intercept fish passing upstream along the cofferdam and direct them into the trap. Attraction water was provided by pumps. As fish were trapped, they were lifted in hopper bottom tanks and unloaded into specially designed fish tank trucks for transportation upstream of the dam where they were returned to the river.

No record has been maintained of the number of fish using the temporary fish ladder near the east bank. During 1959, however, a total of nearly 17,000 fish were trapped and transported by truck. The arrangement of the trap in the cofferdam is shown in Fig. 14, and a closer view of the hoisting equipment for the trap is shown in Fig. 17.

RESERVOIR

All of the work involved in the construction of a hydroelectric project is not confined to the site of the dam and powerhouse. At Rocky Reach, 35 miles of State Highway and 24 miles of railway had to be relocated while traffic on each was maintained. In addition, 20 miles of existing railway embankment had to be protected or improved to withstand the higher water levels of the new reservoir itself will have a surface area of about 10,000 acres and will inundate close to 6,000 acres of farm, orchard and range lands, and a substantial part of the town of Entiat, Washington. The extent of this relocation work and the work of clearing of the reservoir was a major construction operation in its own right and is not included herein.

MAJOR QUANTITIES INVOLVED IN CONSTRUCTION

Excavation for structures, common	370,800 cu yd
Excavation for structures, rock	573,700 cu yd
Foundation Cleanup for Concrete	67,900 sq yd
Excavation for east bank & cutoff dam, common	2,534,100 cu yd
Excavation for east bank & cutoff dam, rock	26,100 cu yd
Excavation for fills, filter zones, riprap etc.	1,475,400 cu yd
Concrete	1,025,000 cu yd
Portland Cement	991,800 bbl
Forms	4,341,500 sq ft
Reinforcing Steel	35,100 ton
Estimated Cost of Project	$273,100,000

ACKNOWLEDGMENTS

The Rocky Reach Hydroelectric Power Project is being constructed for Public Utility District No. 1 of Chelan County, Washington. District Commissioners are L. J. Richardson, President, Ivan J. Compton, Secretary, and Robert O. Keiser, member. Kirby Billingsley is Manager of the District, and E. C. Metcalf is Chief Engineer. Design of the Project and supervision of its construction is being done by Stone & Webster Engineering Corp., of Boston

with D. N. McCord, F. ASCE, Vice President as Construction Manager, J. H. Boyd, M. ASCE, Project Manager at the site, O. L. Hooper; F. ASCE, Chief Hydraulic Engineer, and C. W. Hubbard, Project Engineer. A joint venture consisting of L. E. Dixon Co., Hunkin Conkey, Arundel Corp., and American Pipe and Construction Co., operating under the name of Rocky Reach Dam Builders, performed Stage I Contract work. The Final Stage General Contract is held by Rocky Reach Contractors, consisting of the same four companies plus the Guy F. Atkinson Co. W. N. Evans, of the L. E. Dixon firm, is Resident Manager for the Contractors. Tullar Power Co., is contractor for electrical work, and the Selby Drilling Co. performed grouting work to provide the grout curtain cutoff in the East Bank Terrace.

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CONSTRUCTION DIVISION

Proceedings of the American Society of Civil Engineers

DISCUSSION

POLYVINYL ACETATE AND PORTLAND CEMENT MORTAR[a]

Discussion by Clara F. Derrington

Clara F. Derrington.[11]—Mr. Howe's interesting paper suggests the desirability of calling attention to other work that has been reported in this connection.

A review of the literature to obtain information relating to polyvinyl-acetate (PVA) emulsions proposed for use in concrete mixtures reveals much interest in its use by a few manufacturers and laboratories. Actually, relatively few laboratory tests have been made. Data derived from all available sources are summarized herein.

The earliest and most thorough investigational work using polyvinyl acetate was reported by Geist, Amagna, and Mellor[12] in 1953. Mortar specimens were made and tested for tensile, compressive, and impact strength, abrasion resistance, bond strengths of mortar to steel and mortar to concrete, corrosion resistance, air entrainment, water adsorption, and coefficient of expansion. In general, polyvinyl-acetate emulsion with a nonionic emulsifier having a particle size of 1 to 5 microns was found more effective than emulsions with particle diameters from 2 to 10 microns. In addition, maximum effectiveness was obtained at a polymer-to-cement ratio of 0.2 without a plasticizer, when cured at a relative humidity of 35%. However, when cured at 100% relative humidity, tensile and compressive strengths of all polymer-containing mortars were lower than those of plain cement mortars. Also, it was found that as the amount of added polyvinyl acetate was increased, the strength of the resulting mortars when cured under water was decreased. For mortar specimens containing a PVA-to-cement ratio of 0.2 the following relations were found:

1. Tensile strengths at 28 days of PVA mortars, cured in a dry atmosphere, were ten times those of normal cement mortars cured in a dry atmosphere, and three times those of normal cement mortars cured under water.

2. The compressive strengths of mortars containing PVA, cured at 50% relative humidity, were only 70% of the strength of plain cement mortar cured in a fog room, but more than three times that of plain cement mortar cured at 50% relative humidity.

3. The modulus of rupture and modulus of resilience of mortars containing PVA cured in a room of 50% relative humidity (RH) were several times greater than those of plain mortars cured in a fog room.

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[a] February, 1960, by Robert T. Howe.

[11] Chemist, Special Investigations Branch, Concrete Div., U. S. Army Engr. Waterwys Experiment Sta., Jackson, Miss.

[12] "Improved Portland Cement Mortars with Polyvinal Acetate Emulsions," by Jacob M. Geist, Servo V. Amagna, and Brian B. Mellor, Industrial and Engineering Chemistry, Vol. 45, April, 1953, p. 759.

4. Mortars with polyvinyl acetate, cured at 100% relative humidity, had about ten times the resistance to abrasion of plain cement mortars cured under the same conditions. In addition, resistance to abrasion of mortars increased with increasing PVA content.

5. Mixtures containing PVA showed greater shrinkage than did plain cement mortars. However, when PVA mortars were placed between two rough concrete surfaces, the mortar mix remained unaffected by any shrinkage, and no cracking was evident.

6. The coefficients of thermal expansion for plain mortars and mortars containing PVA were practically identical, at about 11×10^{-6} per degree Centigrade.

7. For PVA-mortar specimens cured in a dry atmosphere, the strength of the bond to steel was 1 1/2 times, and the strength of the bond to concrete surfaces was ten times that of plain cement mortars cured in a dry atmosphere. The bond strength between mortars and old concrete surfaces of specimens cured in a fog room was practically the same for all mixtures, including plain cement mortars.

8. When cured in a fog room, as compared to plain cement mortars, PVA mortars evidenced at least the same resistance to freezing-and-thawing, and equal or greater resistance to alkalies, organic solvents, and dilute inorganic acids. The PVA mortars were more resistant to concentrated hydrochloric acid (HCl), but much less resistance to boiling water.

9. All mortars containing the polymer entrained at least 2% more air than did plain mortars.

10. PVA mortar was more workable and flowed more easily than did plain cement mortars with the same water-to-cement ratio.

11. When polyvinyl-acetate emulsions are added to portland cement mortars in the amount that will produce optimum physical properties of the mixture, the polymer in the mixture, by virtue of the attraction of the water on the surface of its particles, prevents the water from evaporating too rapidly and allows the mortar to hydrate fully. Microscopic examination indicates that the main strength is probably due to cement-gel structure, reinforced by the PVA in the voids.

Rissel[13], reported (in 1953) that two proprietary brands of "concrete emulsion" that consist essentially of polyvinyl acetate have been used successfully in patching cracks and potholes in concrete roads. Later, in 1954[14], he reported on additional observations of "PCI-concrete emulsion" used in patching concrete; because good performance was observed, recommendations were made for the use of such synthetic resins in the patching of concrete pavements. In 1955, Rissel[15] related further experiences with admixtures of synthetic resin in concrete. Experience gained in actual practice showed that, with a plastic-and-cement mortar, lasting repairs to old concrete are possible if certain working rules are observed. Also, Rissel com-

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13 "A New Method of Repairing Old Concrete Roads," by E. Rissel, Strasseu Autobahn, (in German), Vol. 4, No. 7, 1953, pp. 223-227. Translation: PCA Lit. Research Section, Foreign Lit. Study No. 88.

14 "Patching of Concrete Pavements," by E. Rissel, Forshungsgesellschaft für das Strassenwessen E. V. Schriftenreihe der Arbeitsgruppe Betonstrassen No. 5, 1954, pp. 58-61. Translation: PCA Lit. Research Sect., Foreign Lit. Study, No. 147.

15 "Experiences with Admixtures of Synthetic Resin in Concrete," by E. Rissel, Zement-Kalk-Gips, Vol. 8, No. 10, October, 1955, pp. 355-361, Germany, Verein Deutscher Zement Werke, Tagungsberichte der Zementindustrie No. 12, 1956, pp. 59-73.

mented that laboratory tests that have hitherto been used generally for testing plastic-and-cement mortars should be modified and appropriately adapted to practical conditions.

In 1954, Eastwood[16] stated that the addition of a specially prepared polyvinyl-acetate emulsion to Aerocem, a sprayed foam-concrete coating, increased its tensile strength five-to tenfold, as well as improving the bond, adhesion, and resilience of the coating.

A bulletin[17] published by the manufacturer described the general properties and uses of "Elvacet" polyvinyl acetate-water emulsions. "Elvacet" is the manufacturer's trade name for a group of polyvinyl-acetate resins that are available in three grades. The water emulsions of Elvacet have such uses as adhesives, binders, coatings, and prime-sealers in paints. Water emulsions of Elvacet are mechanically and chemically stable, but should not be allowed to freeze because it may cause phase separation or coalescence of the resin. The emulsions contain a small amount of free acetic acid (the exact amount not specified) and should be stored in wood, glass, or noncorrosive containers. The company's electro-chemical department sponsored investigations of Elvacet 81-900 polyvinyl-acetate emulsions, when incorporated with portland cement sand mixtures, at the Armour Research Foundation, Chicago, Illinois. The conclusions based on these investigations were: (1) addition of 20% Elvacet 81-900 polyvinyl-acetate emulsion based on portland cement greatly improves the bond strength of brick to mortar; (2) mortars containing 20% Elvacet and cured for 7 days at 50% relative humidity exhibit maximum compressive and tensile strength; (3) modulus of rupture is considerably increased in concrete containing 20% Elvacet; (4) workability of the mortar is directly proportional to the amount of Elvacet added up to 25%; above 25% the mortars become sticky and difficult to trowel; (5) mortars containing 10% or less Elvacet show excellent resistance to freezing; (6) incorporation of Elvacet increases shrinkage of concrete during curing; (7) the tensile strength of Elvacet-modified concrete is considerably reduced when immersed in water, dilute acid, or alkali, but is regained upon drying; (8) plasticized Elvacet gives lower tensile, compressive, and bond strength than the unmodified emulsion.

In February, 1956, a patent[18] was issued to William D. Robinson on a mixture of 1 part of PVA to 1 to 10 parts portland cement by weight with pigments, modifiers, or waterproofers added if desired. PVA spray-dried emulsion mixed with portland cement is used in water-dispersable paints for masonry surfaces. The PVA obviates the necessity of wetting the wall before application and promotes better adhesion.

An article[19] published in Dutch, in February, 1956, was reviewed by Van Erp with the following comments:

> "The use of synthetic chemical compounds as concrete admixtures, their effect on concrete, and concrete properties at various proportions of admixtures were reviewed. The most widely used admixture is a dis-

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16 "Aerocem," by John A. Eastwood, Indian Concrete Journal, Vol. 28, No. 8, 1954, pp. 328-333.

17 "'Elvacet' Polyvinyl Acetate Water Emulsions," E. I. Du Pont de Nemours & Co., Vinyl Products Bulletin V3-355.

18 "Polyvinyl Acetate Cement Compositions," by William D. Robinson, (Assignor to E.I. du Pont de Nemours & Co., Wilmington, Del.), U.S. Patent 2,733,955. Patented, February 7, 1956.

19 "Synthetic Admixtures to Concrete (met kunststof gelegeerd beton)," by G. J. Hamer, Cement (Amsterdam), Vol. 8, No. 13-14, February, 1956, pp. 327-330. (Reviewed by John Van Erp, ACI Journal, January, 1957, p. 713.)

persion of polyvinyl acetate, and its function is essentially to coat all crystalline particles in the concrete and form semielastic hinges between them. This explains the deformation capacity and the crack resistance of such concrete. Other properties of such concrete are (1) excellent bonding to old concrete, a reason why it can be used in layers as thin as a fraction of an inch; (2) wear-resistance; (3) good elasticity even at low temperatures; (4) increased flexural and compressive strength. Some applications are road repair, airport landing strips, floors, railroad ties, bridges, and storage tanks. Concrete wearing courses as thin as 1/4 in., bonded to old concrete, have been used successfully."

McCoy[20] in May, 1956, reported on additional PVA tests. His findings would tend to discourage the use of PVA as an integral admixture in concrete if the concrete were to be subjected to normal outside weather conditions or high humidities. His studies indicated that when PVA is used as an integral admixture in concrete, it is capable of contributing considerably to strength if the concrete is cured under dry conditions, but the volume stability of such concrete mixtures in the presence of moisture is questionable. During the investigation, neat-cement bars were made with the following percentages of polyvinyl acetate; none, 7.5%, 12%. and 20%. After an initial curing of 24 hr in a moist cabinet, some of the bars were stored outside for 25 weeks. Others were stored in the laboratory at 50% RH for 4 weeks, then at 100% RH for 21 weeks. The volume change of these specimens was determined, and the results revealed that volume change increased with increasing perventages of PVA . After a few weeks of storage at 100% RH, the bars containing the 12% PVA began warping and cracking, and finally, measurements could not be obtained. For the specimens stored outside, it was noted that the 20% PVA bars began warping at 7 weeks, and the 12% PVA bars at 12 weeks. McCoy points out that these tests were made with neat-cement PVA mixtures, that would tend to accentuate any expansive characteristics when compared with mortar or concrete specimens.

In this investigation, compressive strengths were measured on several cubes made with straight cement, cement + 20% PVA solids, and fine silica + 20% PVA solids. When cured at 50% RH, the compressive strengths of the specimens containing 20% PVA with cement and those containing 20% PVA with silica were greater than specimens with straight portland cement. Also at 50% RH, the silica + 20% PVA cubes were noted to have about the same strength as cubes containing cement + 20% PVA° In all instances, the specimens containing cement + 20% PVA decreased considerably in strength when stored at 100% humidity as compared to plain-cement speciemens stored under same conditions. Also, specimens containing silica + 20% PVA possessed even less strength than cement + 20% PVA when stored at 100% humidity. McCoy suggested that when the specimens are dry-cured very little strength is devoloped as a result of the hydration of cement. Rather, most of the strength is derived from the dehydration of the polymer. This is substantiated by the fact that the dry-cured specimens containing PVA + silica possessed strengths equal to or greater than PVA portland-cement specimens. In addition, when both groups of specimens are placed in moist storage subsequent to dry curing, most of the strength is lost.

[20] Letter to Waterways Experiment Station with summary tabulation of data from tests performed by Lehigh Portland Cement Co., by W. J. McCoy, May, 1956.

Groth-Andersen and Heerwagen, in 1956, reported[21] the results of some laboratory tests and described some practical uses of polyvinyl acetate. Concrete beams were broken and glued together with both pure cement pastes and mortars containing PVA. In all cases, there was a high increase in strength when PVA-fortified mortar was used compared to the strength of the pure cement pastes. A repair of a concrete road made a year earlier was unsuccessful, but the addition of PVA to concrete mortar was asserted to increase the durability of concrete pavement.

In an article published in 1956, Nielsen reported[22] observations of repairs of concrete surfaces in which polyvinyl acetate was used. Patches, that were carried out with adhesive layers of PVA, had loosened from old concrete in about 12 months, the adhesive layer swelling under the influence of water. It is recommended by Nielsen that repairs not be made with adhesive layers of PVA , but that the PVA he added directly to the mortar.

Other references[23,24,25,26] promoting the use of polyvinyl acetate in concrete were found in this review of literature, but they merely reviewed the original work performed by Geist, Amagna, and Mellor and offered no new test data.

From this review of the relevant published information and comments concerning the use of polyvinyl acetate as a chemical admixture in concrete, it is apparent that there are differences of opinions concerning the capabilities of mortar containing PVA, and that precise data are lacking.

In every investigation, curing at 100% relative humidity or underwater curing resulted in weaker mortars when PVA had been added. The test data developed by McCoy showed this phenomenon more effectively than did the data from other laboratories, probably because the test specimens made by him were neat-cement pastes rather than cement-sand mixtures.

From these considerations, it appears that mortars containing PVA might be most suitable where curing conditions are maintained at ordinary temperatures and low humidities. Some applications for which this modified concrete may be particularly advantageous and practical are as floor toppings, masonry surfaces, limited structural uses, and patching or bonding of indoor concrete surfaces, because self-curing would be the easiest curing and the surfaces would not be subjected to outside weather conditions and high humidities. There are some references that claim successful patching of cracks and potholes in concrete roads. Other references cite examples of unsuccessful repairs of concrete surfaces using mortar containing PVA. Humidity or presence of water may determine whether the repairs of concrete roads are successful. From the test data, the question arises as to the stability of PVA mortars when exposed to outside weather. This is, particularly significant in heavy rainfall, high humidities, or immersion in water for short periods. It has been stated that concrete containing PVA regains its strength upon dry-

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21 "Polyvinyl Acetate in Concrete," by H. Groth Anderson and K. Heerwagen, Beton-Teknik, Vol. 22, No. 2, 1956, pp. 57-64. (In Danish, with English summary.)

22 "Polyvinyl Acetate for Concrete-water Resistance," by Knud E. C. Nielsen, Beton-Teknik, Vol. 22, No. 4, 1956, pp. 163-164. (In Danish, with English summary.)

23 "Polyco Vinyl Copolymer Emulsions for Self-curing Portland Cement Mixtures, Polyco 470," American Polymer Corp., Technical Data Sheet P-35, March, 1954.

24 "Concrete Admixtures with Polyvinyl Acetate," Industrial and Engineering Chemistry, Vol. 44, No. 5, 13A, May, 1952.

25 "Material of Engineering Construction Review: Cements," by C. R. Payne, Industrial and Engineering Chemistry, Vol. 45, No. 10, 1953, pp. 2185-2188.

26 "Polyvinyl Acetate Emulsion as an Addition to Cement," Cement and Lime Manufacture, Vol. 28, No. 4, July, 1955, pp. 50-51.

ing, but this would not be possible if the concrete cracked, warped, or disintegrated when under the influence of water.

It is apparent that the water resistance of PVA mortars needs to be increased. The Du Pont report[17] indicates that addition of glyoxal, hexylene glycol, ethylene or diethylene glycol monoethyl ether, or some chromium compounds to PVA mortar will improve the water resistance of the film. However, test data could not be found to substantiate the effects of these compounds on mortars containing PVA.

In listing the characteristics of PVA emulsions, they state that the emulsions contain a small amount of free acetic acid. Because acetic acid is known to be injurious to concrete, the PVA might have a detrimental effect upon the hydration of cement. Work has not been done to determine the effect of free acetic acid in PVA on the strength of the PVA mortars. Nagin, Nock and Wittenwyler have reported[27] the following:

> "The knowledge that most existing tractive surface treatments soon lost their initial skid resistance under heavy traffic loads and quite frequently exhibited poor adhesion in thin layers, especially to portland cement concrete, led Reliance to undertake a study of synthetic resins for binders in this use. . . . The first interest of Reliance in developing a surfacing composition came as a result of many years of experience with light-weight concrete bridge floors. It had been observed that the major cause for deterioration of these floors was the cupping action caused by the unequal wear of concrete and steel which leaves hollows between the metal reinforcements where the concrete has worn away. . . .
>
> "The first type of surfacing composition studied was essentially a concrete to which a polymer latex was added. In 1951 and 1952, Coatings Engineers, Inc., Pittsburgh, Pa., investigated the use of neoprene latex, lumnite cement, and mineral fillers. The neoprene-base concretes were generally unsatisfactory. In 1953 a contract was given to the Frankline Institute Research and Development Laboratories of Philadelphia, to study concretes containing polyvinyl acetate latex, using as a reference work done by Amagna, Mellor, and Geist at the Massachusetts Institute of Technology.
>
> This research program yielded concretes that, when dry, were stronger and more resistant to wear, had lower water permeability, and showed better adhesion to base materials and greater skid resistance than conventional concretes. Unfortunately, although these concretes were impermeable to water, they were found to wear bery rapidly when wet. Experiments were continued with the polyvinyl acetate concretes using both cross-linking agents for the vinyl resin and a sodium methyl siliconate to try to reduce the water sensitivity. The results, while rea-

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27 "The Development of Resinous Skid-Resistant Surfaces for Highways," by H. S. Nagin, T. G. Nock, and C. V. Wittenwyler, Highway Research Board Bulletin 184, May, 1958.

sonably good, were not considered completely satisfactory. Attempts to develop a suitable resin-base concrete were then abandoned. . . ."

ACKNOWLEDGMENTS

The material for this discussion is taken from a report[28] issued in July, 1958, prepared under the direction of Leonard Pepper and Thomas B. Kennedy for the Office of the Chief of Engineers, United States Army.

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PHOTOGRAPHIC ANALYSIS FOR CONSTRUCTION OPERATIONS[a]

Discussion by Robert F. Borg

ROBERT F. BORG,[2] F. ASCE.—The method of photographic analysis presented by Mr. Fondahl should provide a rewarding technique to construction management interested in some of the fundamental problems of our industry. Too often, as the author points out, we are content to simply let habit take its course, and permit operations to proceed along the path of least resistance, without seeking to improve them. The specific applications possible with the techniques described in the paper are only hinted at by the author, and this writer, would like to present, herewith, a broader view of this field.

Mr. Fondahl concentrates most of his thought and his technique on essentially small scale, individual, team, or gang activities. There is much work still to be done in this field, and it can be done both in the laboratory and on the job. Surely, we in the industry are all interested in more bricks to be laid per day, more cubic yards of concrete placed per shift, more square feet of tile placed per day per setter and helperand so forth. It is a pity that the author has not provided some actual photographic illustrations and their analyses, but it is hoped that in his closure he may be able to do this.

The writer's experience, from 20 yr in the construction industry, indicates that the basic problem lies elsewhere than in increasing the efficiency of the individual tasks. Essentially, the efficiency sought by the author would result not only through analyzing the rudimentary components of the various individual trades, that by providing continuity for the various trades. It is this continuity that is basic to efficient construction, and perhaps more decisive than whether it is 500 bricks per day or 550 bricks per day, for example, that a bricklayer can lay. Certainly, few activities undertaken by man exceed, in physical size, a modern construction job, and not many products of his physical work exceed it in logistics and complexity.

A building operation requires the close coordination of between 40 and 60 subcontractors and suppliers, and scores of men and tools, most of which are in one way or another interdependent. The manipulation of these forces, so as to provide the correct cues for each trade, each mechanic and each piece of equipment or material is the problem really worthy of study and one to which we hope Mr. Fondahl will next direct his attention. What does it matter how many square feet of forms a gang can erect in one day if there aren't enough footings poured in advance to erect on, or if the plumber isn't ready for us to close his pipe trench under the footings, or if the reinforcing details were completed too late for bars to have arrived at the job, or if unexpected rock has slowed down the excavation, and so on, and on. In every one of these

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[a] May, 1960, by John W. Fondahl.

[2] Genl. Mgr., Kreisler-Borg Constr. Co., White Plains, N. Y.

interrelated trades is the core of analysis of construction operations. Surely, Frank Gilbreth, as the author pointed out, turned from analyzing "bricklaying techniques" to more "fruitful fields", because there are indeed more fruitful fields in the broader aspects of analyzing field operations.

Just as the skilled aerial surveyor can examine photographs of terrain and derive intelligence and knowledge, so can the skillful analysis of memo-motion pictures and micro-motion pictures reveal consequential data. The memo-motion pictures, described by the author, had 3 sec intervals between pictures. What happens to the results when the intervals are stretched to 30 days between pictures? The accompanying illustrations, (Figs. 1-18) make the results startlingly evident. It enables us to study a broader view of the project operations. "Broader" means that overall construction planning and techniques are analyzed to provide the proper scheduling and coordination of the various parts of the work. This is done primarily in order to yield the greatest amount of unbroken stretches of continuity in the work for the greatest number of trades and men.

The accompanying illustrations were progress photographs made by a professional photographer during the course of construction of a building project by the firm with which the writer is associated. The job shown is that of the construction of Public School 27, Yonkers, N. Y. This building, the total approximate cost of which is $893,390.00 consists of a thirteen classroom and two kindergarten addition to a small existing school building. The building also has a large octagonal shaped all purpose room, kitchen, stage, library, faculty room, and offices. The building is constructed on a reinforced concrete foundation with a structural steel and steel joist frame. The floors are poured concrete on corruform, the roof pre-cast lightweight concrete planks. The building features aluminum windows and entrances, terrazzo and asphalt tile floors, and glare reducing tempered glass.

From an inspection of these figures, we may learn the following:

1. Was the contractor's chosen sequence of operations for demolition effective? (West to East, or back of photograph towards front)the result was the efficient removal of old buildings and debris.

2. Did the foundation sequence that was chosen result in the desired ultimate completion sequence? (The owner desired that the classroom wing and boiler room be ready for occupancy before the rest of the building. The classrooms are in the two storey wing in the rear and in the one storey wing at left. The boiler room is in the angle left in the old building.) Foundations were started in the classrooms first (May, 1959). The steel was erected first in the classrooms (June, 1959). The boiler room area was delayed, however, and the steel was not erected until August, 1959.

3. Was access road for rock excavation (lower right, May, 1959) best, in view of the circumstances? The route seems shortest and most direct, particularly for access by materials trucks after the start of the classroom superstructure in (July, 1959).

4. The steel joist erection in June, 1959 was performed by stacking joists in bundles at various levels. Does this method give the best result? This makes the most efficient use of a crane for hoisting into place. Bundles are landed on floors quickly, the crane is dismissed, and the joists are separated by hand using only two men per joist.

FIG. 1.—DEMOLITION OF EXISTING BUILDINGS COMMENCED, 3% COMPLETE.

FIG. 2.—DEMOLITION NEARS COMPLETION, SITE GRADING UNDERWAY. 9.5% COMPLETE.

FIG. 3.—FOOTINGS COMMENCED ON CLASSROOM WING, ROCK BEING EXCAVATED ALL PURPOSE WING. 13% COMPLETE.

FIG. 4.—FOUNDATION WALLS AND WATERPROOFING UNDERWAY IN CLASSROOM WINGS, FOUNDATIONS COMMENCED BOILER ROOM. 19.5% COMPLETE.

FIG. 5.—STRUCTURAL STEEL AND JOIST ERECTION PROCEEDING CLASSROOM WING, FOUNDATIONS BOILER ROOM COMPLETE, FOUNDATIONS COMMENCED ALL PURPOSE WING. 29.5% COMPLETE.

FIG. 6.—ROOF DECK AND CONCRETE FLOORS INSTALLED CLASSROOM WING, WATERPROOFING OF FOUNDATIONS COMPLETE, STEEL DOORS AND FRAMES DELIVERED. 35% COMPLETE.

FIG. 7.—ALL STRUCTURAL STEEL AND BAR JOISTS ERECTED, ROOF DECK, CONCRETE ON CORRUFORM, AND SLABS ON GROUND, BEING COMPLETED, MASONRY, CARPENTRY, STEEL STAIRS, AND DRAINAGE COMMENCED. 41.5% COMPLETE.

FIG. 8.—ROOFING COMMENCED, ALUMINUM WINDOWS DELIVERED. 50% COMPLETE.

FIG. 9.—EXTERIOR WALLS UNDERWAY, WINDOWS BEING INSTALLED, PLASTERING UNDERWAY. 60% COMPLETE.

FIG. 10.—EXTERIOR WALLS COMPLETE, GLAZING AND CALKING COMMENCED. 71% COMPLETE.

FIG. 11.—MASONRY AND GLAZING COMPLETED, ROLLUP GRILLES INSTALLED, WOOD FLOORING, CHALKBOARDS, HARDWARE COMMENCED, TEMPORARY HEAT IN OPERATION. 78% COMPLETE.

FIG. 12.—MILLWORK, TILE, TERRAZZO, FURNISHINGS AND EQUIPMENT COMMENCED. 87.5% COMPLETE.

FIG. 13.—RESILIENT FLOORS, PAINTING, COLD GLAZED ENAMEL, TOILET STALLS, VENETIAN BLINDS AND ALUMINUM RAILING COMMENCED. PLASTER AND TERRAZZO COMPLETE. 93.5% COMPLETE.

FIG. 14.—OUTSIDE WALKS COMMENCED. TOILET ACCESSORIES INSTALLED. CLASSROOMS OCCUPIED. 95% COMPLETE.

FIG. 15.—OUTSIDE WALKS COMPLETED, TOPSOIL DELIVERED. 97% COMPLETE.

FIG. 16.—OUTSIDE RAILINGS AND FLAGPOLE INSTALLED, TOPSOIL SPREAD. 99% COMPLETE.

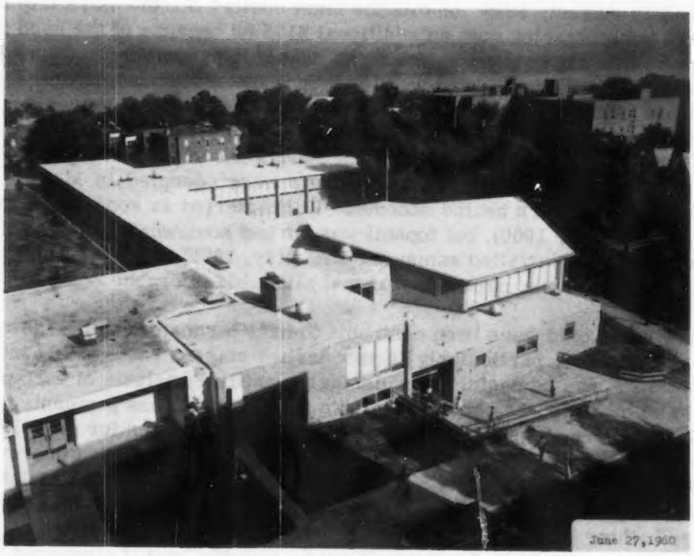

FIG. 17.—LAWNS AND PLANTING COMPLETE. 100% COMPLETE.

FIG. 18.—COMPLETED PROJECT.

5. The lightweight pre-cast concrete roof deck was done in two trips in July, 1959, and in August, 1959. Is this the least costly? No; the extra trip by the roof deck subcontractor cost an additional $175.00 because of the need for an extra move for his equipment, men, and crane. It was evidently unavoidable, however, if the classroom wing was to be kept ahead of schedule.

6. What effect did winter weather have on the job from November, 1959 through March, 1960? The outside site work is at a virtual standstill, class rooms are enclosed, the roof is tight, and work is proceeding on interior finishes using temporary heat. Classrooms were occupied in March, 1960.

7. Was the site work behind schedule? Walks started as soon as the ground was thawed (April, 1960), but topsoil was spread somewhat late (May, 1960); outside rails were installed somewhat late (May, 1960); lawn and landscaping was installed within a regional seasonal allowable tolerance (June and July, 1960).

Clearly, it would have been desirable to have a shorter interval for some of the photographs, particularly in the earlier stages. The monthly interval used, however, can be useful in furnishing a quantitive analysis of each photograph by comparing the photographs with monthly progress payments. As an example compare Figs 6 and 7 with the percentage gains for the following phases of work for the 27 day period from July 20 to August 24: Concrete and Cement 24%; Reinforcing Steel 5%; Structural Steel 1% ; Open Web Joists 5%; Masonry 35%; Roof Deck 83%: Carpentry 20%; Miscellaneous Metals 60%; Furring, Lathing & Plastering 1.5%; Drainage System 30%.

A quantitative analysis, by item, if made monthly, and if compared to the monthly memo-motion photos, could yield information of vast use to the construction engineer.

In summarizing, a photographic analysis of construction operations as proposed by Mr. Fondahl, can be useful in examining crews and methods in which micro-motion and memo-motion pictures ranging in intervals of 1,000 frames per minute to 3 sec are used. It can also be useful when, as proposed by this writer, intervals of greater duration up to for example, 30 days are chosen. It is in the latter type of studies that the construction engineer gains information relating to the broader aspects of project mangement, and job coordination.

Mr. Fondahl is to be congratulated for bringing a basic tool to the attention of the profession.

Acknowledgments.—Photographs by H. Bernstein used with permission of the architect, Eli Rabineau, and the owner.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

VOLUME 85 (1959)

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248 (SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

VOLUME 86 (1960)

JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411 (SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436 (ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445 (EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454 (EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478 (HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487 (CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502 (EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511 (EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520 (PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529 (HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580 (EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596 (HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

OCTOBER: 2615(EM5), 2616(EM5), 2617(ST10), 2618(SM5), 2619(EM5), 2620(EM5), 2621(ST10), 2622(EM5), 2623(SM5), 2624(EM5), 2625(SM5), 2626(SM5), 2627(EM5), 2628(EM5), 2629(ST10), 2630(ST10), 2631(PO5)[c], 2632(EM5)[c], 2633(ST10), 2634(ST10), 2635(ST10)[c], 2636(SM5)[c].

NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

c. Discussion of several papers, grouped by divisions.

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Your attention is invited

NEWS OF THE CONSTRUCTION DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

DIVISION ACTIVITIES
CONSTRUCTION DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

October, 1960

CONSTRUCTION EDUCATION QUESTIONNAIRE

The last few pages of this Newsletter contain a questionnaire that all Construction Division members are urged to fill out. Its purpose is to assist the Joint Committee of the American Society for Engineering Education (ASEE) and the Associated General Contractors of America (AGC) in determining what men in the construction industry desire as training for engineering graduates seeking employment with construction firms.

A slightly modified version of the questionnaire has already been sent to 6,000 contractors through the National Committee on Construction Education of AGC. The first returns indicate an unusual type of response and interest in such a study. The educators involved from ASEE have commented that these returns were quite revealing and rather different than anticipated. Thus, the Joint Committee requested ASCE's assistance in polling CD members to obtain a greater sampling.

Aside from assisting engineering colleges in shaping their curricula in construction engineering, questionnaire results will be compiled in a form that would guide high school students interested in construction engineering in choosing a school and curriculum.

MAMMOTH POOL PROJECT ON PHOENIX PROGRAM

The Job Planning Committee will conduct a full session at the ASCE Phoenix convention next April on the Mammoth Pool Dam Project. It will deal with the planning required by the various engineers and contractors involved in the project.

To be coordinated by Neville S. Long, the session will feature speakers representing Bechtel Corp., Morrison-Knudsen Co., Utah Construction Co. and the Consolidated Western Steel Co., reports committee chairman Roy G. Cappel.

EQUIPMENT MAINTENANCE SURVEY

The Equipment Maintenance Committee has announced that the response to its nationwide survey of the needs of the construction industry in the field of

equipment maintenance has been very favorable. Returns have been received from more than 50% of the firms queried.

The presidents of 170 major engineering and construction firms were asked their problems and needs concerning equipment maintenance. Replies have recently been tabulated and committee members are now interpreting the results to draw meaningful conclusions for later dissemination to the Construction Division and ASCE as a whole.

CD SECTION CONDUCTING SURVEY

Surveys continue to be the order of the day, this time by the San Francisco Section's Construction Division. The CD organization is looking into local contract administration practices.

Progress to date appears to warrant expansion of the survey and a request for funds has been made to the CD Committee on Research and also to the ASCE Executive Committee, reports Joseph Kaplan, San Francisco CD chairman.

The San Francisco CD, one of the most active local CD's, has been asked to prepare a summary of its "modus operandi" for the benefit of other local section construction divisions.

PERAINO RECEIVES CONSTRUCTION ENGINEERING PRIZE

The CD Executive Committee has approved the recommendation of the Construction Engineering Prize Committee that Joseph Peraino be awarded the Construction Engineering Prize at the Boston convention in October for his article "Glen Canyon Dam." The article appeared in the June 1959 issue of Civil Engineering.

There will be no second order of merit award.

SURVEY COMMITTEE HAS A SURVEY, TOO

Admiral Lewis B. Combs, chairman of the Committee on Construction Industry Surveys, reports that Questionnaire No. 5 recently has been sent out. Replies are being received and evaluated by James T. Norton, advertising manager of Civil Engineering. Purpose of the committee is to work up raw material through surveys of a panel of 250 to 500 engineering construction executives to indicate the function of the civil engineer as a purchasing authority.

CONTRACTS COMMITTEE COMPLETES PROJECT, MISSES DEADLINE

The Contract Documents Committee completed its review and report on the ASCE-AGC Form of Contract (heavy construction) but failed in its effort to delay publication of the 1960 edition so that its recommendations could be incorporated. The ASCE-AGC Form of Contract is normally revised every few years.

Because the Contract Documents Committee feels a number of provisions in the contract form can be improved in accord with its recommendations, the

committee is now pushing for a very early revision of the 1960 edition, which has just been published. Committee chairman Blair Birdsall is hopeful in this regard.

COMBINED INDEX TO ASCE PUBLICATIONS

For complete coverage of the Society's 1959 year in print, there is now a Combined Index covering the Division Journals, Transactions, and Civil Engineering. Also included are reprints of the Proceedings Abstracts that are published each month in Civil Engineering. The price of the Combined Index (ASCE publication 1960-10) is $2.00 with the usual 50% discount to members. The coupon herewith will make ordering easy:

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CONSTRUCTION EDUCATION QUESTIONNAIRE

The Joint Committee of the American Society for Engineering Education (ASEE) and the AGC as well as the National Committee on Construction Education of the AGC strongly urge you to answer the following questions aimed at finding out what those interested in construction want in Education for Construction. The information gained from your replies will be of great assistance in guiding the work of the above committees.

Please detach and return to Walter L. Couse, 12740 Lyndon Avenue, Detroit 27, Mich.

1. Does your firm give preference in your employment policy to graduates of a school with an accredited curriculum in civil or architectural engineering?
Yes__________ No__________.

2. Would the inclusion of construction subjects in the civil or architectural engineering curriculum encourage your employment of more engineers?
Yes__________ No__________.

3. If a five year curriculum including all the essentials of a civil or architectural engineering four year course plus education for construction management (patterned somewhat after courses in buisness management) leading to a BSCE and a Master of Construction Engineering degree were available, would you prefer graduates so educated rather than by a standard four year course?
Yes__________ No__________.

4. What type of college level training of prospective construction engineer employees do you consider most important to the construction industry. (Answer by giving 1st, 2nd, and 3rd choices.)

__________ a. Engineering design
__________ b. Construction management (See #3)
__________ c. Construction operations, plant layout, field operations, etc.
__________ d. General business management

5. Do you favor including management and broad education courses in the college curriculum such as:

__________ a. Humanities
__________ b. Economics
__________ c. Accounting
__________ d. Estimating and Cost Accounting
__________ e. Banking and Corporation Finance
__________ f. Labor-management Relations and Labor Law
__________ g. Business Reports and Public Speaking
__________ h. Real Estate and Engineering Law

Yes__________ No__________.

If your answer is "Yes" will you rate the above eight subjects in the order of your preference. Use separate sheet for further discussion.

6. Do you favor the inclusion of specific construction subjects in the proposed "Construction Engineering" curriculum such as:

_____ a. Construction planning
_____ b. Construction materials
_____ c. Heating, plumbing, air-conditioning, electrical installation
_____ d. Soil mechanics, earthwork and foundations
_____ e. Construction estimates and costs
_____ f. Formwork design
_____ g. Site layout and development
_____ h. Contracts and specifications
_____ i. Engineering valuation (appraisals, depreciation, insurance, taxes, etc.)

Yes_____ No_____.

If your answer is "Yes" will you rate the above nine subjects in the order of your preference.

Any other areas of construction study desired (use separate sheet for discussion)

7. There is a strong movement, particularly by the AIA, to abandon the curriculum known as architectural engineering. If you are interested in Building Construction do you have any preference in your employment policy between a civil and architectural engineer.

Yes_____ No_____.

If the answer is "Yes" which is your preference._____

8. Please comment on "What sort of training you would desire in engineering graduates who seek employment in the Construction Industry" (use separate sheet).

9. Submit any other ideas you may have to help the Committee on Education (use separate sheet).

Name_____ Title_____

Firm_____ Type of work firm performs_____

Address_____ _____

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